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THESIS TITLE

Seismic Retrofit of Reinforced Concrete Frame Buildings with Tension Only Braces

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Thesis submitted to the University of Ottawa
in partial fulfillment of the requirements for the

Doctor of Philosophy

in

Civil Engineering

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EXECUTIVE SUMMARY

Reinforced concrete buildings built prior to the enactment of modern seismic codes are often seismically deficient. These buildings may have inadequate strength and ductility to withstand strong earthquakes. Conventional retrofit techniques for such frame buildings involve adding reinforced concrete shear walls or structural bracing systems to the existing bays. These techniques can be intrusive and result in lengthy down times and expensive structural interventions. An alternative to conventional techniques is the use of high-strength prestressing strands or cables, diagonally placed as tension elements. This technique was researched and used in a limited manner after the 1985 Mexico City Earthquake. It has since been further investigated at the University of Ottawa through experimental and analytical research (Shalouf and Saatcioglu (2006), Carrière (2008), Molaei (2014)). While the use of steel strands as tension bracing elements proves to be an effective technique, the resulting stiffening effects on the frames lead to increased seismic force demands and higher based shear, as well as increased axial forces on the attached columns, potentially generating net tension, foundation uplift and excessive compression. Relatively low elongation characteristics of high-strength cables and slack caused by yielding strands and associated pinching of hysteresis curves reduce potential energy dissipation capacity. The current research aims to improve the previously observed deficiencies of the system. One of the improvements involve the use of shape memory alloys (SMA) in the middle of the cables, which reduce/eliminate residual deformations upon yielding and associated pinching of the hysteresis curves. SMA allows energy dissipation in the system while forcing the structure to recover from its inelastic deformations because of the flag-shape hysteretic characteristics of the material. The feasibility of the cable-SMA assembly as tension brace elements is illustrated through dynamic analyses of selected prototype buildings. The other improvement is the development of progressively engaging, initially loose multiple strands as tension cables. These cables are placed loosely to engage in seismic resistance at pre-determined drift levels, thereby eliminating premature increase in seismic force demands until their participation is required as the frame capacity is reached. Tests of a large-scale reinforced concrete frame, designed following the requirements of the 1965 National Building Code of Canada NRC (1965) as representative of existing older frame buildings

in Canada, are conducted under simulated seismic loading to assess the effectiveness of the proposed system. The verification of the concept is extended analytically to prototype buildings and the effectiveness of the system is demonstrated for mid-rise and low-rise frame buildings.

ACKNOWLEDGEMENTS

I would like to express my deep and sincere gratitude to my research supervisor, Dr. Murat Saatcioglu for giving me the opportunity to do research and providing invaluable guidance and support throughout this research. It was a great privilege and honor to work and study under his guidance. I am extremely grateful for what he has offered me. I would also like to thank him for his friendship, empathy, and great sense of humor.

I am very much thankful to my wife Maryam Sohrabi and my daughter Luna, for their love, understanding, and continuing support to complete this research work. I am extremely grateful to my parents and brothers for their love, prayers, caring and sacrifices for educating and preparing me for my future.

Many thanks to lab technical officers Dr. Muslim Majeed and Dr. Gamal Elnabelsy for their support and help during my lab work. The help and support of Faraz Ahi Moghaddam, is appreciated during the process of construction and setting up the test frame.

Finally, my thanks go to all the people who have supported me to complete the research work directly or indirectly.

Sadegh Khosravi

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CHAPTER 1

1 Introduction

1.1 Background

Existing reinforced concrete buildings designed and built prior to the enhancement of modern seismic codes were essentially designed to resist gravity and wind loads without much emphasis placed on seismic design and detailing. Inelastic response and ductility of these buildings were not considered in design with sufficient details. Hence, they exhibit non-ductile or partially ductile response when subjected to strong earthquakes. Progressively improved seismic provisions of more recent design codes after the 1970s promote dissipation of seismic induced energy through ductile flexural yielding and plastic hinging in critical regions of elements. Buildings designed following the requirements of recent codes are expected to perform essentially in the elastic mode of deformations during more frequent minor earthquakes with limited or no damage. They are expected to develop repairable damage when subjected to moderate earthquakes and suffer significant damage while maintaining life safety without collapse when subjected to rare but strong earthquakes. As design provisions in building codes have improved over the last four decades, seismic hazard predictions also improved, reflecting advances in the field of seismology. During this period, more ground motion records have become available. Furthermore, improved attenuation relationships have been developed. These improvements led to better assessment of design force levels. Seismic design force requirements have increased by more than 100% relative to those of the pre-1970 era. Moreover, more strict drift limits were imposed to control seismic damage. Clearly, older buildings remain to be seismically deficient, both from the design force perspective, as well as the design and detailing requirements for individual elements.

1.2 Research needs and significance of research

It is economically not feasible to replace seismically deficient buildings with new buildings designed to have improved seismic resistance. Therefore, seismic retrofit remains to be a viable seismic risk mitigation strategy. Retrofitting is done either at the element level, to improve performance of otherwise seismically deficient elements, or at the global level to enhance overall lateral load resistance and drift control. The former is implemented if

seismic deficiency is limited to a few elements, and the latter is preferred if a large segment of the lateral load resisting system is deficient. In the latter case, the structure benefits globally from overall strengthening and stiffening of the entire system. This results in drift control, which reduces seismic damage. Bracing is an effective method for increasing lateral stiffness and controlling drift which has been widely researched and used in practice. Many numerical and experimental research projects have been undertaken in the past, dating back to 1970's, or some even earlier. Among the early researchers, Higashi and Kokusho (1975), Higashi et al. (1977), Kawamata and Ohnuma (1981), and Sugano et al. (1980, 1981, 1983) showed that bracing is a proper method of repairing and strengthening concrete structures. At the time, adding shear walls to concrete moment resisting frames was a common and preferred retrofit technique for storey drift control. Lateral bracing can be done either by adding shear walls or steel bracing elements of different types and orientations.

One of the lateral bracing systems consists of steel cables or strands. In this system the brace elements can only resist tension. Tension braces are placed symmetrically in two directions so that either the braces in one direction or the other direction develop tension and provide resistance to seismic loads. Pincheira and Jirsa (1995) investigated the effectiveness of tension braces in buildings. They compared the technique with adding shear walls to existing concrete frame buildings. The researchers concluded that the tension bracing system can be an effective strategy for seismic retrofitting. This technique was researched and used in a limited manner after the 1985 Mexico City Earthquake. It has since been further investigated at the University of Ottawa through experimental and analytical research Shalouf and Saatcioglu (2006), Carrière (2008), and Molaei and (2014). While the use of steel strands as tension bracing elements proves to be an effective technique, the resulting stiffening effects on the frames lead to increased seismic force demands and higher based shear, as well as increased axial forces on the attached columns, potentially generating net tension, foundation uplift and excessive compression. Relatively low elongation characteristics of high-strength cables and slack caused by yielding strands and associated pinching of hysteresis curves reduce potential energy dissipation capacity. One of the improvements proposed to the tension brace system involves the use of shape memory alloys (SMA) in the middle of the cables, which reduces or eliminates residual deformations

upon yielding and associated pinching of the hysteresis curves. SMA allows energy dissipation in the system while self centering the structure to recover from its inelastic deformations because of the flag-shape hysteretic characteristics of the material. Advantages of using SMA are shown analytically as part of the investigation. The frame elements, as well as the diagonal cables, with and without SMA are modeled to simulate inelastic response using computer software “SeismoStruct (2016), which is equipped with the appropriate hysteretic models for both reinforced concrete and SMA elements. However, the use of SMA in structural applications can be costly. Furthermore, partial use of the material within the tension cables poses connection/coupling problems at the SMA ends. Therefore, another technique is also investigated as an improvement to tension cable braces. The second improvement involves the use of progressively engaging cables (PEC). Accordingly, the strands are placed loosely as tension elements such that each strand engages in resisting lateral forces progressively, when needed. In this application, the strands initially remain unstressed until the lateral drift becomes sufficiently high, and the frame requires further lateral resistance. As the first strand yields and dissipates energy, the second and possibly the third strand starts engaging in lateral load resistance, while some strands remain elastic, recovering some of the inelastic deformations. The brace cables do not initially engage in lateral resistance (at drifts of less than 1%) and remain loose, thereby avoiding unnecessary stiffening and associated higher seismic forces until the stiffening effects of cables are needed to control the lateral drift. The strands can be designed to limit lateral drift to the deformation capacity of the frame so that it continues fulfilling its gravity load carrying function while the tension braces provide increasingly more resistance to lateral loads. This type of retrofitting is investigated through combined experimental and analytical research. Tests of a large-scale reinforced concrete frames, designed following the requirements of the 1965 National Building Code of Canada NRCC (1965) as representative of existing older frame buildings in Canada, are conducted under simulated seismic loading to assess the effectiveness of the proposed system. The verification of the concept is extended analytically to prototype buildings. Research on this topic is scarce in the literature and limited to the application to structural steel frames. Additional research is needed to fully develop the system for non-ductile reinforced concrete frame buildings.

Combined experimental and analytical research has been conducted to fill the research gap.

1.3 Objectives

The primary objective of the research project is to develop economically viable and structurally effective tension bracing systems to retrofit reinforced concrete frames against seismic forces. More specifically, the objective includes improvements in existing cable bracing systems by incorporating SMA elements in tension braces and by developing progressively engaging cables (PEC), both for reduced seismic force demands, pinching of hysteretic response, and increased energy absorption capacity, inelastic deformability and drift control, while promoting self centering capabilities for the structure.

1.4 Scope

The scope of proposed research includes the following tasks:

- Review of previous research on seismic retrofitting with emphasis on bracing of frames against seismic loads.
- Selection of a prototype reinforced concrete building frames for experimental and analytical research.
- Selection of a scaled reinforced concrete frame test specimen and appropriate retrofit schemes for testing.
- Selection of computer software for non-linear response time history analysis of the test frame, as well as conducting analytical parametric investigation.
- Analytical modelling of the selected test frame with and without different types and configurations of cable bracing systems for dynamic inelastic analysis with specific emphasis placed on material hysteretic behaviour for SMA and prestressing tendons.
- Experimental research and tests of a large-scale reinforced concrete frame with different configurations of progressively engaging strands for validation of the concept.
- Validation of the effectiveness of SMA as part of the tension brace system in selected

prototype buildings.

- Validation of the effectiveness of using progressively engaging prestressing strands as part of the tension brace system in selected prototype buildings.
- Compilation of experimental and analytical research and presentation of results.

1.5 Presentation of Thesis format

The thesis is presented in “manuscript-based” format. It consists of 6 Chapters. Three chapters include three different manuscripts, hence may include some repetition of basic concepts and literature review for completeness. The details of each chapter are described below:

- **Chapter 1:** Includes introduction where research needs, and significance of research are summarized. It also includes objectives and scopes of the research program.
- **Chapter 2:** Presents “Literature Review,” where different seismic retrofit techniques for non-ductile reinforced concrete frame buildings investigated by previous researchers are reviewed. The literature review emphasises on using tension-only braces as an effective practical method for controlling lateral drifts. The limited previous research on the application of SMA as part of concrete frame bracing system and progressively engaging braces for frames is presented.
- **Chapter 3:** Presents the journal article “Seismic Retrofit of Non-Ductile Concrete Frames with Cable-SMA Assembly,” to be submitted to *Earthquake Spectra*. Structural design and analysis of a 2-storey and 6-storey non-ductile RC buildings in Vancouver based on the 1965 edition of the National Building Code of Canada (NRCC 1965) is presented. The effectiveness of the Steel-SMA assembly as tension bracing system is demonstrated on selected mid-rise and low-rise prototype buildings.
- **Chapter 4:** Presents the journal article “Progressively Engaging Cables as Tension Braces for Seismic Retrofit of Non-Ductile Concrete Frames,” to be submitted to *ASCE Journal of Structural Engineering*. The new system of lateral bracing, consisting of progressively engaging cable (PEC) system is presented with

experimental and analytical components. The experimental component includes two tests of a large-scale reinforced concrete frame with two different arrangements of the PEC system. Analytical component includes non-linear analysis of the test frame retrofitted with different configurations of the PEC system and their validations against the experimental results.

- **Chapter 5:** Presents the journal article “Seismic Retrofit of Reinforced Concrete Frames by Progressively Engaging Tension Cable (PEC) Braces,” to be submitted to the Canadian Journal of Civil Engineering. It presents the application of the PEC system of retrofitting to 2-storey and 6-storey prototype buildings selected in Vancouver, Canada, designed based on the 1965 edition of the National Building Code of Canada (NRCC 1965). The chapter demonstrates superior performance of the PEC system.
- **Chapter 6:** This chapter presents summary and conclusions of the research project.
- **Appendices:** There are three appendices, presented as supplementary information that is complementary to the main body of the thesis. These include design data for the prototype that represent the scaled test frame, additional experimental data generated during the tests and the selection and analysis of another set of prototype buildings for Ottawa, Canada. In addition to the two prototype buildings selected and designed for Vancouver, a 2-storey and a 6-storey building was also designed and analyzed for Ottawa. However, these buildings showed acceptable performance under the set of earthquake records that are compatible with the 2015 NBC (NRCC 2015) and hence did not need seismic retrofitting. Therefore, they are not included in the main body of the thesis, which focused on the seismic retrofit methodologies implemented. In summary, three Appendices are included, consisting of; i) Appendix I - Design details for the building representing the scaled test frame, ii) Appendix II – Design and dynamic analysis of selected prototype buildings for Ottawa, Canada, iii) – Additional test data recorded.

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CHAPTER 2

2 Literature Review

2.1 Overview of seismic retrofit techniques

Seismic retrofit of existing concrete structures is implemented either at the element level, as retrofitting of columns, beams, beam-column joints or walls; or globally, affecting the entire lateral load resisting system by adding lateral bracing for strength, stiffness and drift control, or by changing the dynamic characteristics of buildings through the use of supplementary damping or base isolation devices.

At the element level, the emphasis is often placed on improving seismic force-deformation characteristics of columns, which are responsible for overall strength and stability of frames. Column jacketing is a standard retrofit scheme for seismically deficient non-ductile columns. This is done through using reinforced concrete, steel, or fibre reinforced polymer (FRP) jackets. A large number of experimental and analytical studies were conducted on column retrofitting. Chai et al. (1991, 1994, 1996) conducted extensive research on steel jacketing of circular and rectangular columns. Other researchers who conducted research on steel jacketing includes Xiao and Wu (2003), Li et al. (2005), Priestley et al. (1991, 1994a, 1994b, 1996). These research programs led to widespread implementation of steel jacketing to bridge infrastructure in California. Reinforced concrete column jacketing was also researched extensively by Rodriguez and Park (1994). Another material which has become popular in recent years for column jacketing (column wrapping) was FRP. FRP jacketing was researched by Saadatmanesh et al. (1994, 1996, 1997), Seible et al. (1995), Xiao et al. (1997), Pantelides et al. (1999), Gergely et al. (1998), and Elnabesity and Saatcioglu (2004, 2018). Use of transverse prestressing for columns was also developed for active column confinement. Extensive testing and analytical research were conducted at the University of Ottawa for developing an externally applied transverse prestressing technique for concrete columns (Saatcioglu and Yalcin (2003), Saatcioglu et al. (2001)). In addition to the extensive column research programs, beam-column joints have been observed to suffer joint shear failures during past earthquakes and retrofit techniques were developed for the joints. Moehle and Mahin (1991), and Said and Nehdi (2004) conducted research on retrofitting beam-column joints to avoid local failures in joints. These regions of frames can be enhanced by increasing joint shear capacity. This can be done by providing

additional reinforced concrete, steel or FRP sleeves at the joints. Beams of non-ductile frames often perform as flexure-dominant members and they too benefit from local enhancements of potential plastic hinge regions and/or through shear strengthening if they have shear deficiencies, using the same construction materials.

System level seismic retrofit techniques are implemented to affect global structural behaviour of the lateral load resisting system. Global retrofit techniques often involve introducing lateral braces to the structure or enhancing the existing structural or non-structural bracing elements to improve their behaviour under seismic loading. The proposed research project involves the development of new tension bracing systems for seismically deficient frame buildings. Therefore, the emphasis on the literature review is placed on tension braces with an overview of other global retrofit systems, including those that are aimed to change the dynamic properties of the structure by implementing supplementary damping devices or base isolation units, as well as active and passive control systems. The following sections are devoted entirely to the review of global retrofit systems used in practice or in previous research projects.

2.2 Lateral bracing

The proposed research involves enhancements of tension brace systems. Therefore, the emphasis on literature review is placed on global interference by lateral bracing systems; more specifically, lateral bracing with steel tension elements. However, the use of more conventional bracing systems involving the addition of shear walls, as well as enhancing existing walls with surface-mounted FRP or steel strips is also presented briefly for completeness. Furthermore, the problems associated with steel bracing, in terms of their effects on frame elements, is similar to those caused by adding new shear walls. Hence, they are presented in the following two subsections without attempting to summarize the extensive literature that is available on these topics.

2.2.1 Use of shear walls as bracing elements

Adding reinforced concrete shear walls to existing concrete moment resisting frames has been used in practice during the last three decades and has become a conventional seismic retrofit strategy. It is often used to strengthen non-ductile frames. It is especially well suited

to reinforced concrete frame structures. Compatibility of the existing and new materials and relatively easy integration of the walls in the enclosing frame elements in selected bays offer advantages. In this method, the maximum available ductility in the un-retrofitted structure is first assessed, and new shear walls are added such that the overall drift capacity remains within this limit after the incorporation of the new shear walls. This ensures that the frames continue fulfilling their functions as gravity load elements when they go for the ride during seismic response. The wall reinforcement can be well integrated into the enclosing frame elements by drilling holes into the beam and column concrete, and epoxy bonding the wall reinforcement into the surrounding elements through these holes. Fig. 2-1a shows a typical application in practice with reinforcement cage placed between the enclosing frame elements. The challenges of adding shear walls in existing buildings include limitations in the architectural plan to place them, and heavy demands imposed on the foundation. The part of the foundation located under these additional walls may have to be reinforced and retrofitted, as the walls become responsible from resisting a large portion of lateral loads with a tendency to lift up. Furthermore, building new reinforced concrete walls is an intrusive process, increasing the down time during construction. The addition of concrete structural walls has been researched extensively in the past and was shown to be an effective seismic retrofit methodology (e.g. Altin et al. (1992); Arslan et al. (2010); Canbay et al. (2003); Frosch et al. (1996))

While the addition of reinforced concrete shear walls is most suited for concrete frame systems, steel plate shear walls are also used as lateral bracing elements in structural steel frames. In this case their main function become resisting horizontal story shears and overturning moments cause by lateral loads. In general, steel plate shear wall systems consist of a steel plate wall, two boundary columns and horizontal floor beams. Fig. 2-1b illustrates a steel plate shear wall system. The use of steel shear walls is rare in RC buildings.

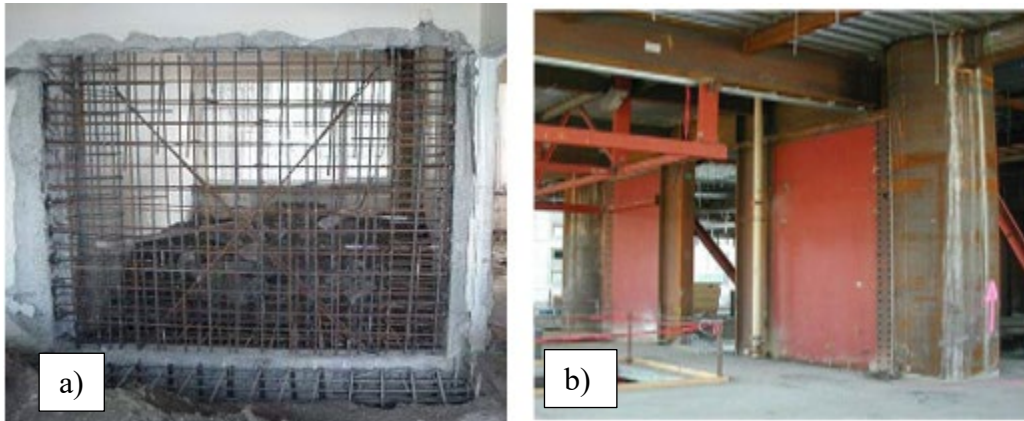


Figure 2-1 Adding a) concrete b) steel shear walls as retrofit technique

2.2.2 Strengthening existing structural and non-structural walls with FRP and steel strips

It is a common practice to use infill masonry walls in older buildings, as well as in some of the more recently built structures. These walls are non-structural elements, and they are deficient in strength and ductility. However, when used in frames, covering entire bays, they do interfere with structural response and start providing diagonal compression resistance. Hence, they provide an excellent opportunity to serve as shear walls, if properly retrofitted. The use of surface mounted FRP sheets, aligned either in two orthogonal directions or placed diagonally provide diagonal tension resistance, improving the wall behaviour. If integrated with the enclosing frames, they serve as shear walls and they control lateral drift, thereby increasing the lateral force resistance of the entire frame-infill wall assembly. Fig. 2-2 shows the application of carbon FRP sheets to infill concrete masonry walls in a reinforced concrete frame with the FRP secured to the enclosing frame elements (Saatcioglu et al. (2005), Saatcioglu (2019)).

Load bearing masonry walls also benefit from FRP retrofitting. Surface mounted FRP sheets increase diagonal tension capacity of walls. When combined with the high diagonal compression capacity of masonry, they can function as shear walls, providing increased lateral force resistance. However, this system does not offer any appreciable ductility as both the masonry and the FRP are brittle materials. Therefore, such retrofitting strategy must be based on elastic or limited inelastic response, which may be possible in most buildings

as the long horizontal dimensions of these walls are often sufficient for increased in-plane resistance if diagonal tension capacity is increased with FRP. The increase in in-plane resistance can be further enhanced by attaching the already provided surface-bonded FRP to the supporting elements of the walls, in which case the FRP acts as flexural reinforcement. Alternatively, FRP retrofitted masonry walls can be secured to the supporting element (foundation wall for first storey walls) by means of FRP anchors or ductile steel sheet anchors (Arifuzzaman and Saatcioglu (2012); Saatcioglu (2019)). The application of FRP diagonal braces on existing masonry infill walls was also investigated by Binici et al. (2007), Erdem et al. (2006), and Ozden and Akguzel (2006). Other aspects of masonry wall retrofit by surface-bonded FRP was researched extensively in the past (Ehsani et al. (1997) Ehsani and Saadatmanesh, 1996 and 1997; Triantafillou Thanasis (1998); ElGawady Mohamed et al. (2007); Willis et al. (2009); Mahmood and Ingham (2011); and Aiello and Sciolti (2006)).



Figure 2-2 Adding a) concrete b) steel shear walls as retrofit technique

Load bearing concrete masonry walls with surface-bonded FRP sheets can also be reinforced internally by ordinary re-bars that can provide flexural resistance (Sabri (2013)). A similar retrofit technique can be implemented through prestressing the walls to their supporting elements with the added benefit of self centering capability (Sabri (2020)). Tests of unreinforced masonry load bearing walls retrofitted by internally placed re-bars and prestressing strands are illustrated in Fig. 2-3. Other researchers who investigated prestressed masonry include Laursen and Ingham (2001, 2004), Rosenboom and Kowalsky

(2004) and Levy and Spira (1973).

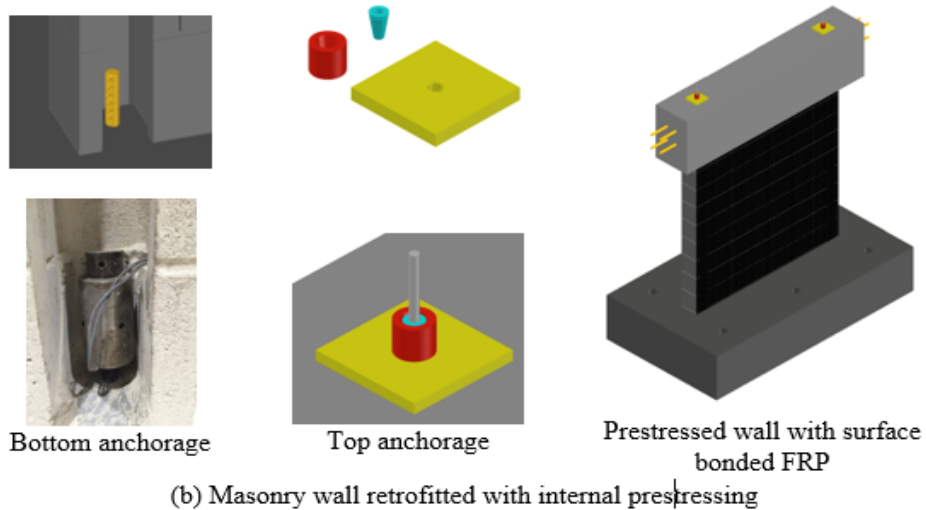
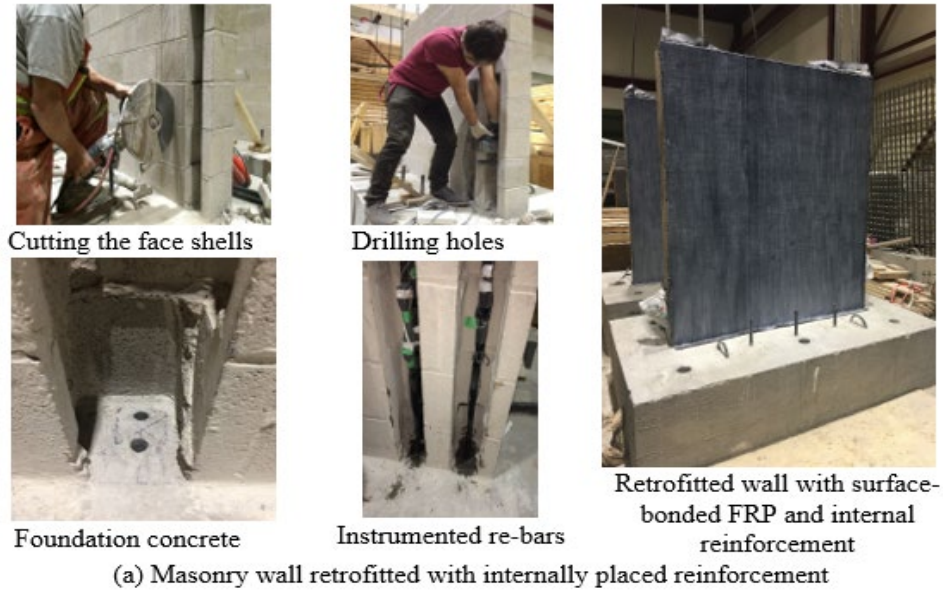


Figure 2-3 Masonry walls internally reinforced or prestressed for seismic retrofitting

Another method of improving strength and ductility of load bearing masonry walls, and non-ductile reinforced concrete walls, is the use of externally placed steel strips. These strips can be secured on opposite sides of walls and provide flexural reinforcement, as well as diagonal tension and compression resistance. Taghdi et al. (2000) conducted extensive experimental and analytical research on the topic and showed that externally placed steel strips can improve strength and ductility of otherwise brittle walls significantly, serving as

reliable seismic bracing elements. Fig. 2-4 shows retrofitted masonry and non-ductile reinforced concrete shear walls.



(a) Masonry wall



(b) Reinforced concrete wall

Figure 2-4 Seismic retrofit of brittle wall by steel strips (Taghdi et al. 2000a)

2.2.3 Steel braces in frame buildings

Extensive survey of previous literature was completed on the use of steel braces in enhancing seismic behaviour of frames. These techniques include the use of concentric and eccentric structural steel bracing, compression bracing, buckling restrained braces (BRB), and tension braces with and without SMA's. The literature on these categories are presented below with more emphasis placed on tension braces, which provide the scope for the proposed research project.

2.2.3.1 Regular structural steel braces

Structural steel braces are used to provide additional lateral force resisting capacity and drift control for structures. Diagonal bracing forms tension ties and compression struts, which resist lateral forces. In some cases, braces are designed to remain elastic and in other cases they are expected to yield and dissipate seismic energy. It is more economical to allow braces to develop plastic deformations and dissipate energy during strong earthquakes. However, braces that develop plastic deformations experience permanent deflections and the structure may not return back to its original position. Therefore, these braces are often designed to remain elastic. Fig. 2-5 shows the application of X-braces in practice as a

retrofitting strategy as implemented in a building at Shiraz University of Medical Sciences in Iran.

The behavior of diagonal tension and compression braces was investigated extensively in the past. Maheri and Sahebi (1997) tested four small-scale concrete frame models by applying monotonically increasing diagonal compression forces. The test program included an un-retrofitted frame, a braced frame with tension-only diagonal brace, a braced frame with diagonal compression-only brace, and finally a braced frame with tension-compression X- brace. The test frames had a 580 mm square opening and a member thickness of 100 mm. These frames were designed based on the ACI 318-63 (1963) code using the ultimate strength method. All frames were positioned under a universal compression testing machine while making 45-degree inclination with the horizontal. This ensured the application of the compression force along the diagonal. Fig. 2-6 illustrates the geometric dimensions of the test specimens and the test setup.

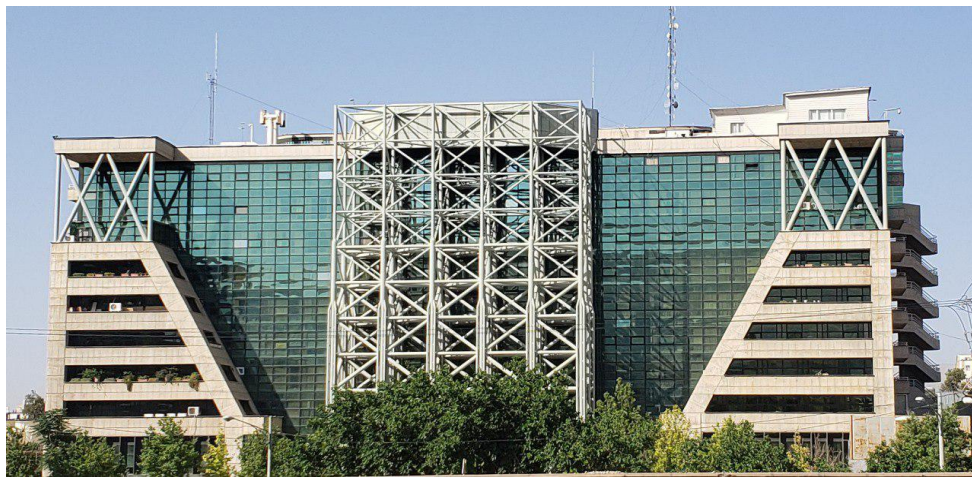


Figure 2-5 Using externally placed steel braced frames as seismic retrofit methodology for Shiraz University of Medical Sciences, Iran

The results indicated that the frames braced with a single tension or compression brace developed increased in-plane shear strength by a factor of 2.5 irrespective of whether they were loaded in diagonal tension or compression. When an X brace was used (two braces one in tension and the other in compression) the in-plane shear capacity increased by a factor of 4 relative to the un-retrofitted bare frame, with 15% higher load being resisted by the

tension brace at ultimate. Failure of all four frames started with flexural distress at four frame corners followed by the loss of load bearing capacity. However, the initiation of failure resulted in braces reaching their peak capacities. This implies that the frame with a compression only brace failed through compression buckling. The frame with a tension-only brace suffered weld failure at the corners. The frame with an X- brace suffered from a plate fracture in the welded connection at mid-length of the tension brace. This was followed by the buckling of compression brace. The researchers subsequently investigated the effectiveness of selected connection arrangements in enhancing the capacity of the frame for new and existing buildings. They tested the connection models illustrated in Fig. 2-7. The authors indicated the importance of extending their research to develop more appropriate anchorage systems that allowed braces to develop their full capacities prior to experiencing local failures in the connections.

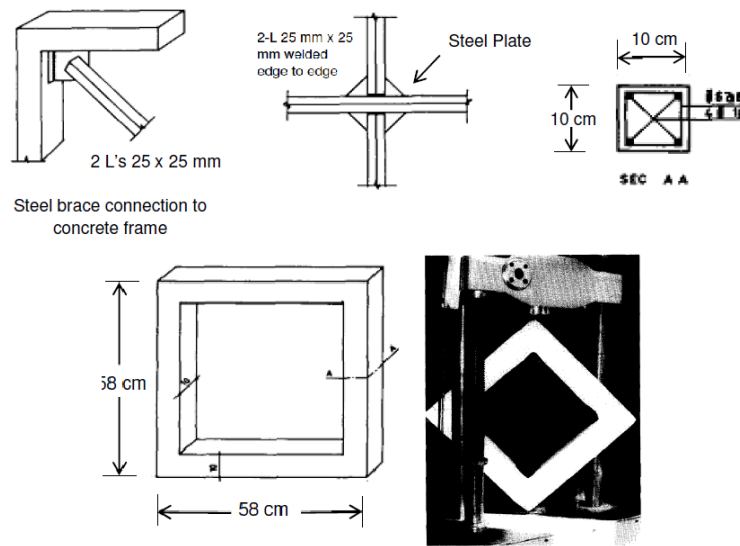


Figure 2-6 Test specimen setup (Maheri and Sahebi 1997)

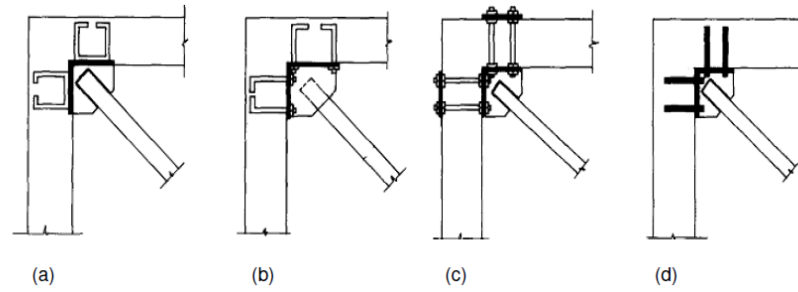


Figure 2-7 Possible scenarios for brace connections: (a) & (b) for frames under construction; (c) & (d) for existing frames (Maheri and Sahebi 1997)

Caron (2010) investigated experimentally the benefit of utilizing compression-only diagonal braces in retrofitting non-ductile RC frames. Two damaged, half-scale frames were repaired and tested as shown in Fig. 2-8. The frames were designed based on the ACI 318 (1967) building code, which represented the design practice prior to the implementation of modern seismic design requirements. The frames were repaired and one of the frames was tested without any retrofitting under simulated seismic loading as the control frame (BR-3R). The other frame was retrofitted with compression-only diagonal HSS braces (BL-3R). Though the frame had an X-brace, it was not connected to the frame to develop tension resistance.



(a)



(b)

Figure 2-8 Test specimens: a) repaired RC bare frame BR-3R; and b) repaired and retrofitted RC frame BL-3R (Caron 2010)

The results showed limited ductility of frames. The unretrofitted and retrofitted frames

developed approximately 2% and 3% lateral drifts, respectively. The control frame developed significant damage at the base of the columns. The retrofitted frame sustained damage in the foundation and showed out-of-plane-buckling. Braces did not reach their maximum load capacity. The results demonstrated that the lateral load capacity of the retrofitted frame increased to 5 times the bare frame capacity. At the same time, the initial stiffness at the beginning of the test increased up to 7 times relative to the un-retrofitted frame. Fig. 2-9 illustrates the envelop curves obtained from the two tests. The author suggested filling of the HSS brace with concrete for increased stiffness of the frame and for prevention of brace buckling.

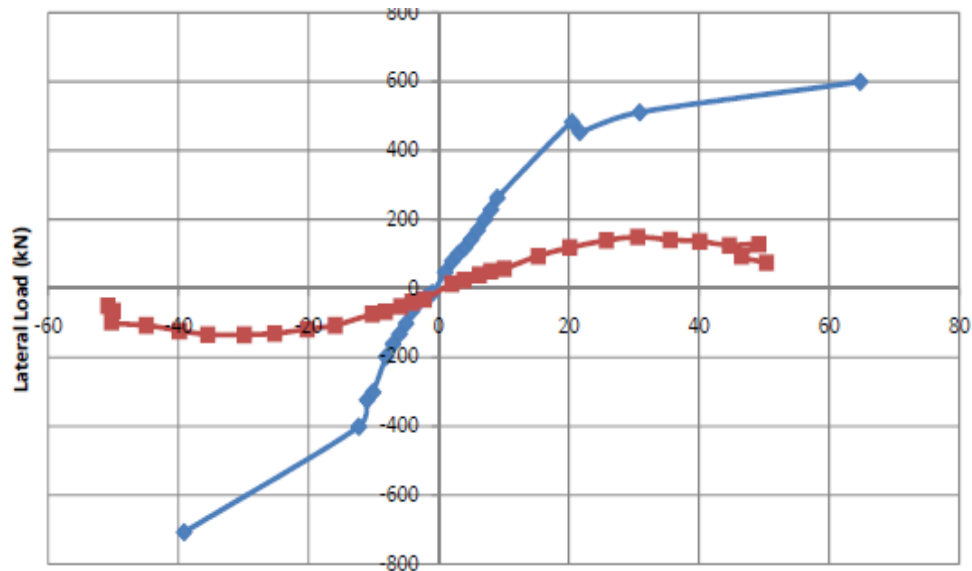


Figure 2-9 Lateral load Vs lateral displacement responses (Caron 2010)

Bush et al. (1991) conducted experimental research on a 2/3rd scale, three-story, two-bay frame subjected to static push over cyclic loads at the third floor. The original frame was built with ductility and strength against seismic loads. It was retrofitted by connecting exterior wide flange X braces, extending two floors continuously. The X braces were attached to a steel frame consisting of vertical steel channels and horizontal T section collectors. The steel frame was attached to the columns in the concrete frame along the height as shown in the Fig. 2-10. The results indicated that the retrofit strategy resulted in both strength and stiffness increases under lateral loads. The lateral load capacity increased up to 2.24 times the design capacity and 6 times the calculated failure capacity. Failure of

the frame initiated by buckling of the braces followed by the failure of welded brace connections. Subsequently, the frame suffered from column shear failure. This research focused on the connection quality, which was mentioned as the main reason for the brace failure.

Sahoo and C. Rai (2009) used a damper which was made of aluminum, in combination with steel braces to act as a shear-yielding link damper (Al-SYD), as a passive energy dissipation device to retrofit and increase the seismic capacity of non-ductile reinforced concrete frames. From the point of deficiency of strength in the columns, they were confined by steel cages as shown in Fig. 2-11. The main purpose of the research was to improve the stiffness, lateral load capacity and overall energy dissipation of the entire frame. The test specimen consisted of a scaled single-span, single-storey reinforced concrete frame with a scale factor of 1:2.5.

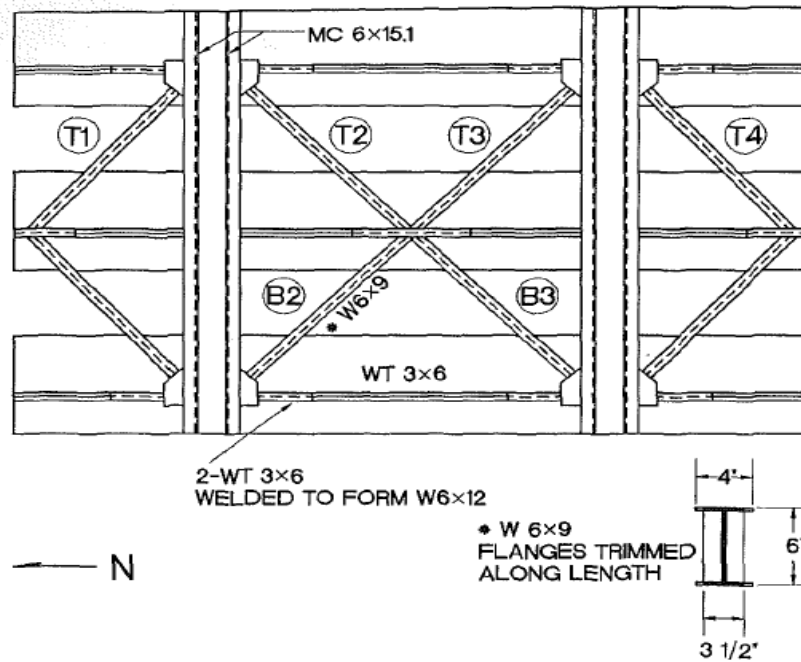


Figure 2-10 Model bracing scheme

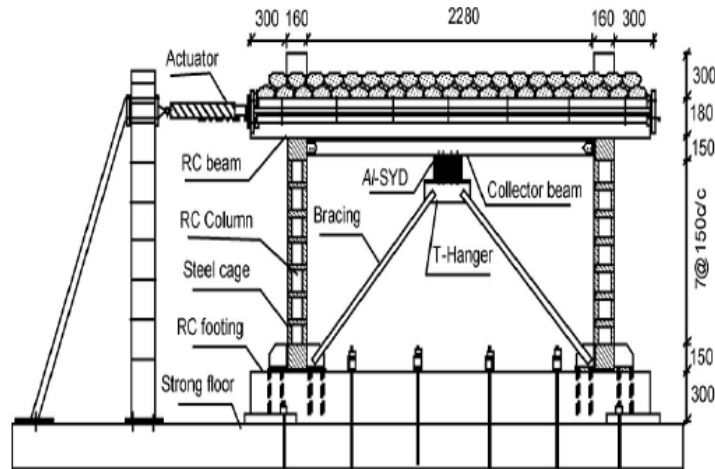


Figure 2-11 Schematic diagram of test set-up (Sahoo and Rai 2010)

The frame was loaded by gravity loads before the horizontal actuator applied lateral force cycles using cyclic lateral displacements as per ACI-374.1-05 (loading protocol (ACI-374.1-05 (2005))). A collector beam was used to transfer a portion of the lateral load to the passive energy dissipation devices, as shown in Fig. 2-12. The frame initially yielded at 0.5% drift ratio. The maximum drift of the frame was almost 3.5%. The lateral load capacity of the retrofitted frame was considered as the sum of the column steel encasements and the contributions of the Al-SYD system. The Al-SYD was able to successfully dissipate considerable energy during the test. The comparison of the hysteretic response of retrofitted and un-retrofitted bare frames is depicted in Fig. 2-13.

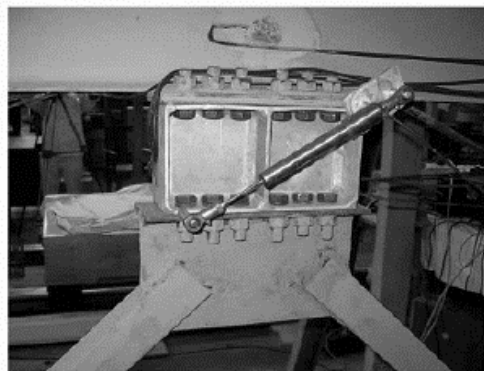


Figure 2-12 RC frame retrofit by Al-SYD link damper (Sahoo and Rai 2010)

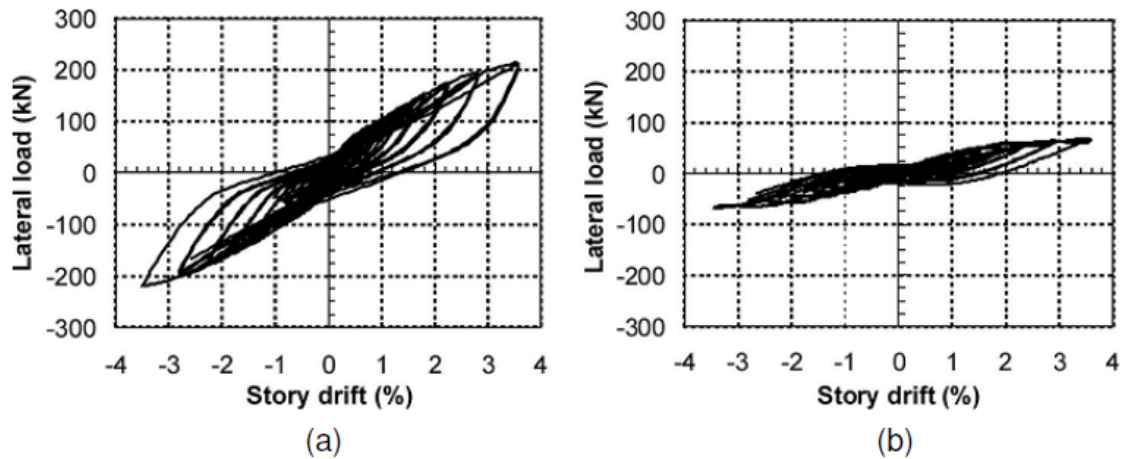


Figure 2-13 Hysteretic response of retrofitted frame: (a) Steel jacketing and AL-SYD; (b) Steel jacketing only (Sahoo and Rai 2010)

The braces and the collector beam remained within the elastic range in both tension and compression. No other type of failure, such as failure of the welds or pull-out of the anchor bolts was observed at 3.5% of drift ratio.

Youssef et al. (2007) tried to increase the efficiency of steel braces in reinforced concrete frames by reducing the mass of the building and increasing its ductility through experimental research. Two frames were designed and tested. The 1st frame was an un-retrofitted moment-resisting frame, moderately ductile with details based on the ACI-318-02 (2002) concrete code. The 2nd frame was designed as frames with concentric internal braces as shown in Fig. 2-14. The frames were 2/5th scale of the original frame and included $150 \times 120 \times 10$ mm gusset plates at each corner, anchored to the concrete frame with four-16 mm diameter rods. All the braces were designed on the basis of AISC and LRFD. The braces were checked according to the AISC seismic provisions for steel structures. Fig. 2-15 shows the test setup. The un-retrofitted frame failed through the formation of plastic hinges caused by yielding of bottom beam bars. These plastic hinges at the end of the beam made it unstable and led to the failure. No shear cracks were observed, except for some flexural cracks in the columns. The use of braces added lateral stiffness to the frame. The crack pattern in the connection area under cyclic loading is shown in Fig. 2-16.

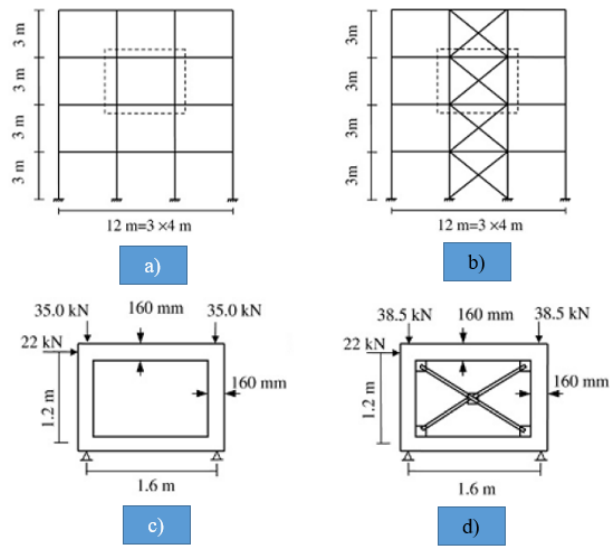


Figure 2-14 a) Moderately ductile un-braced frame b) Braced frame c) Scaled moderately ductile RC moment frame d) Scaled brace frame

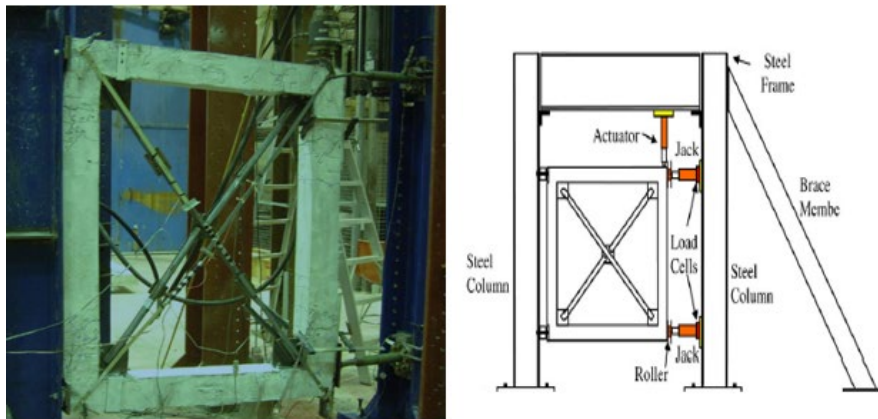


Figure 2-15 Lateral load resisting frame specimen setup (Youssef et al. 2007)

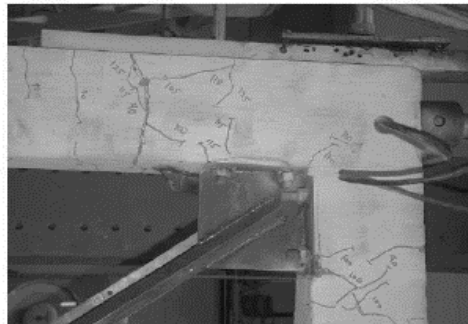


Figure 2-16 Crack pattern in the braced frame connection region

The initial stiffness of the frame was almost 2 times the stiffness of the un-retrofitted moment resisting frame. The lateral load capacity of the retrofitted frame also increased by a factor of 2.6 relative to the bare frame. At higher lateral drifts, the first symptoms of failure were observed in the brace elements when the brace started yielding. The concrete frame showed fewer cracks with signs of the initiation of plastic hinge. The failure initiated by buckling of braces and the formation of plastic hinges at both the ends of the beam. The maximum drift ratio for the braced frame before failure was 4%. Fig. 2-17 illustrates the hysteretic relationships for the two frames. The researchers recommended further research before a design procedure could be developed.

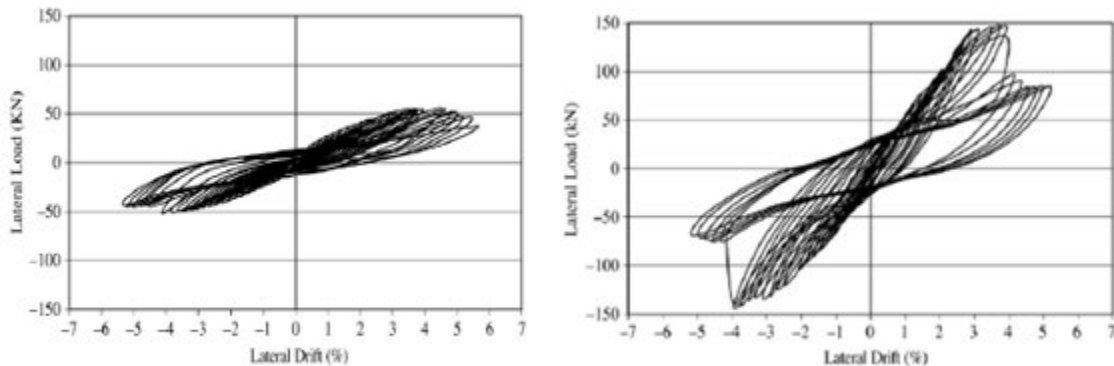


Figure 2-17 Hysteretic behavior of unbraced RC frame (left); Retrofitted RC frame (right)

Maheri and Ghaffarzadeh (2008) continued the above research project of Youssef et al. (2007) by introducing some modifications to the bracing system. The main objective was to determine the level of interaction between the capacities of the reinforced concrete and the brace system. They designed and built another specimen similar to that tested by Youssef et al. (2007) with the same dimensions, designed based on ACI-318-02 (2002). This time the frame was braced with non-slender imperial channel cross sections (C3 x 3.5) compared with the slender braces used in the earlier tests. This frame yielded at 2.5% and failed at 4.3% of drift ratios. The comparisons of the results showed that the new frame failed at a substantially increased lateral load capacity of 200 kN. The researchers measured the displacement diagonally along the braces to find strains in braces so that they could find forces in the brace elements. The capacity of this frame was less than the summation of the

capacities of the braces and the bare frame individually. This was attributed to the over-strength introduced by reducing the effective lengths of the beams and columns due to the brace connections to the frame.

Ozcelik et al. (2012) investigated the effectiveness of chevron braces in non-ductile reinforced concrete frames through experimental research. The researchers tested seven large-scale retrofitted frames under reversed cyclic loading. Fig. 2-18 shows the experimental setup. The researchers concluded that the chevron bracing system improved strength, stiffness and energy dissipation capacity of the retrofitted frames. The braces essentially behaved in the elastic mode of deformations until they experienced compression buckling. Typical hysteretic relationships for an unbraced and a braced frame are compared in Fig. 2-19.

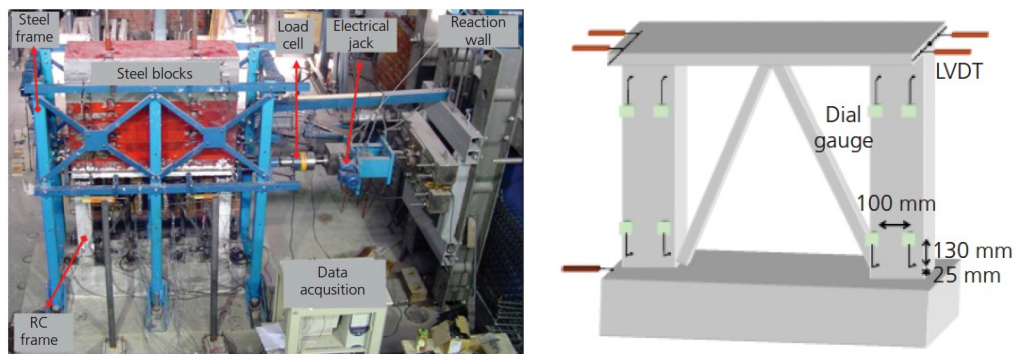


Figure 2-18 Frame retrofit by chevron braces (Ozcelik et al., 2012)

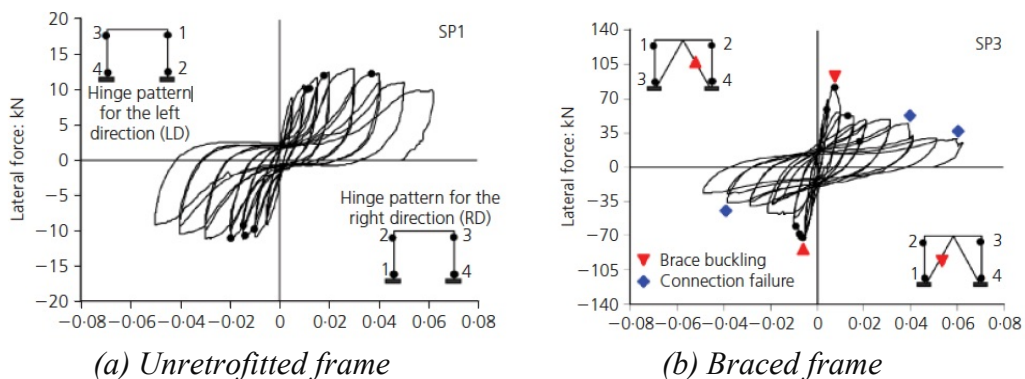


Figure 2-19 Comparison of hysteretic relationships (Ozcelik et al. (2012))

The use of steel braces as seismic retrofit strategy has been investigated by other previous researchers (ex: Badoux and Jirsa (1990); Bartera and Giacchetti (2004); Bush et al. (1991);

Fukuyama and Sugano (2000); Ghobarah and Abou-Elfath (2001); Higashi et al., 1984; Kawamata and Ohnuma (1981); Maheri and Hadjipour (2003); Maheri and Sahebi (1997); Masri and Goel (1996); Mazzolani (2008); Ozelik and Binici (2008); Ozelik et al. (2011); Pincheira José and Jirsa James (1995); Sugano et al. (1980, 1981, 1983); Sugano and Fujimura (1980); Tagawa et al. (1992); Yamamoto and Umemura (1992)). It was demonstrated by previous research that bracing through the use of structural steel members provide lateral strength and stiffness. However, buckling of steel in compression limits inelastic deformability unless prevented.

2.2.3.2 Buckling-Restrained braces (BRBs)

Buckling-Restrained Brace (BRB) is another type of bracing system which can be considered as a special class of concentrically braced frames providing resistance in tension and compression. These types of braces were initially investigated and developed by Watanabe et al. (1988) in Japan. After the 1995 Kobe Earthquake, the researchers in Japan developed BRB as a damper bracing system for seismic retrofitting. Components used in typical BRBs are well explained in the literature (Tremblay et al. 1999, 2006, 2011; Uang and Nakashima (2004); and López et al. (2004)). Fig. 2-20 shows the main components of a typical BRB. The steel core is comprised of three segments: (1) The restrained yielding segment (Section C-C), which can be any type of steel section; ACI-318-02 (2002) the restrained non-yielding transition segment (Section B-B), which is an extension of the restrained yielding segment but with a larger cross sectional area; and (3) the un-restrained, non-yielding segment (Section A-A), which is an extension of the restrained non-yielding segment (Section B-B). Both the restrained yielding and non-yielding segments are housed within the buckling restraining component, while the un-restrained, non-yielding segment extends beyond. The latter provides a connection point for the brace to the frame.

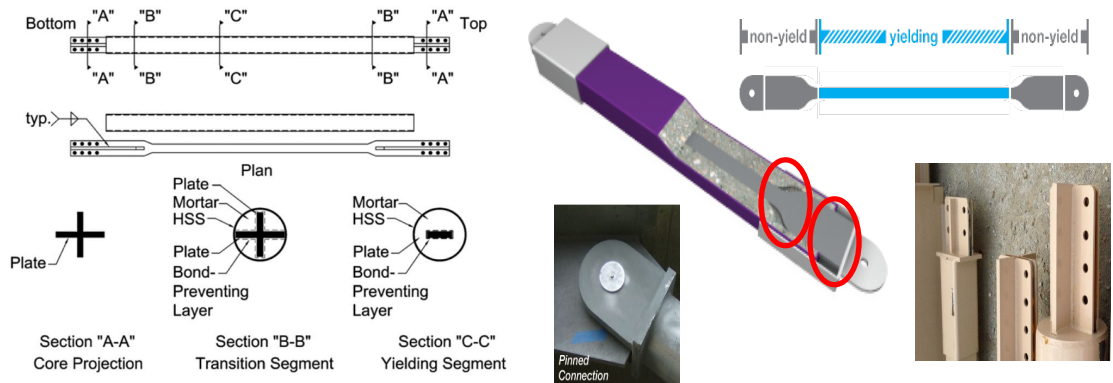


Figure 2-20 Details of a typical BRB

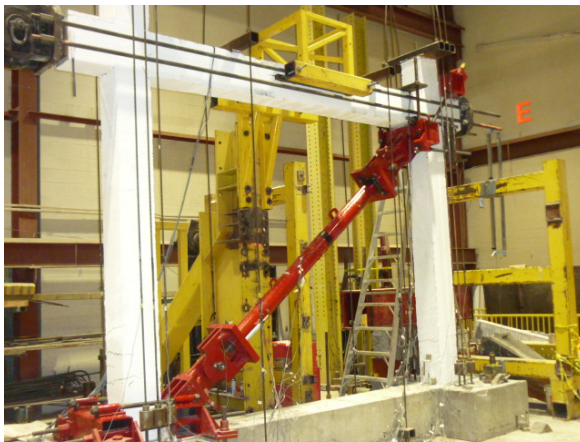
The first usage of BRB in the United States was in 2000 after many large-scale tests conducted at the University of California Clark (2000). By that time the usage of this method in Japan had been increased in a way that there were more than 250 buildings which were retrofitted with BRBs (Uang and Nakashima (2004)). Subsequently, this method of retrofit was employed in more than 350 buildings in the USA (Kimberley and Cameron (2011)). Additional research was conducted on BRBs by Maheri and Sahebi (1997) and Dinu (2011). López et al. (2004) investigated the performance and efficiency of the buildings retrofitted by past BRB systems. Their numerical analysis of RC frames showed advantages of using BRBs. Di Sarno and Manfredi (2009) also conducted a numerical non-linear investigation involving 3D analyses of buildings retrofitted with BRBs. One of the deficiencies of the BRB systems was the failure of braces in the external and internal reserve gaps of the BRB system. These gaps were designed to avoid compression on the encasement and limit the deformations to the core steel element. Due to this gap, the steel core usually failed under out of plane buckling (Tremblay et al. (2006); Aiken et al. (2002); and Zhao et al. (2011)). The other deficiency of the system, which was investigated by Mazzolani (2009) and D'Aniello et al. (2009) was the failure due to a premature fracturing of steel core plates.

The application of BRB systems include bracing both steel and concrete frame structures. One of the first research projects involving concrete frames was conducted by Al-Sadoon (2016) at the University of Ottawa. The research program resulted in the development of a new BRB system without the internal gap and associated internal buckling problem. The

experimental setup and the new BRB are shown in Fig. 2-21. Numerical analyses were used to demonstrate the effectiveness of Buckling Restrained Braces (BRBs).

2.2.3.3 Tension bracing (diagonal tension only) for seismic retrofitting

The literature review presented in the preceding section demonstrated the effectiveness of structural steel bracing providing diagonal tension and compression, while also presenting the shortcomings or challenges of the system in terms of compression buckling, connection details and intrusive nature of the application. An alternative for lateral bracing of seismically deficient frames is tension only brace, which forms the objective of the proposed research project. Therefore, a more thorough review of literature is provided on this topic in two subsections; i) cables and prestressing strands as tension brace members, and ii) shape memory alloys (SMA) as part of tension braces.



Test frame



End unit

Figure 2-21 New BRB system developed at the University of Ottawa with gapless end units (Al-Sadoon et al. 2020)

2.2.3.3.1 Cables and prestressing strands as tension brace members

Ten years after the 1985 Mexico City Earthquake, Pincheira José and Jirsa James (1995) conducted analytical investigations on use of tension cables for retrofitting non-ductile RC frames, focusing on a three-storey, a twelve-storey and a seven-storey building located in high seismic zones in the United States. They investigated the behaviour of the buildings retrofitted by different patterns of post-tensioned braces and X braces compared to the same

buildings retrofitted by infill structural walls and the un-retrofitted buildings. These patterns are shown in Fig. 2-22.

The columns adjacent to the braces were modelled as strengthened columns to avoid failure caused by excessive forces in the braces. The results obtained from the 3-storey building analyses are compared in Fig. 2-23. The results indicate that the use of post-tensioned rods resulted in the highest increase in base shear resistance of the building. The analyses also indicated that the low-rise un-retrofitted frame suffered complete failure under the Mexico City Earthquake while the other records considered indicating deficiencies only at the first-floor level. On the other hand, all three retrofit techniques considered were successful in protecting the building against the same earthquakes. The maximum inter-storey drift was about 1% and it occurred at the second floor of the building retrofitted with post-tensioned bars. The inter-storey drifts for unretrofitted and retrofitted buildings are shown in Figs 2-24 and 2-25, respectively.

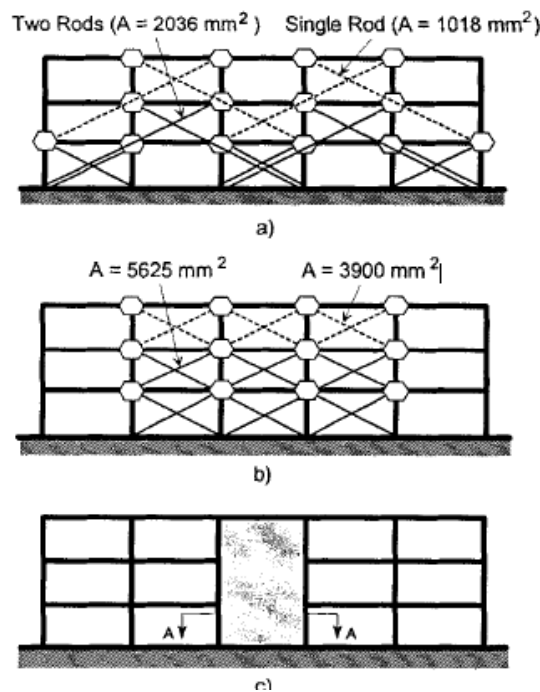


Figure 2-22 Retrofit Schemes for Three-Story Building: (a) Posttensioned Bracing; (b) X-Bracing; (c) Infill Wall

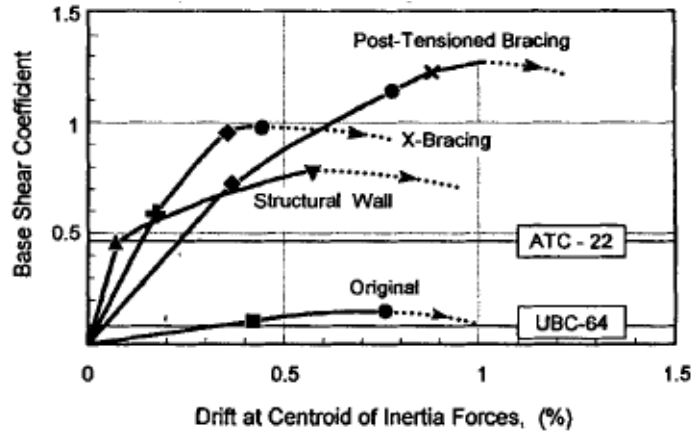


Figure 2-23 Comparison of base shear-drift relationship for original and retrofitted three-story buildings

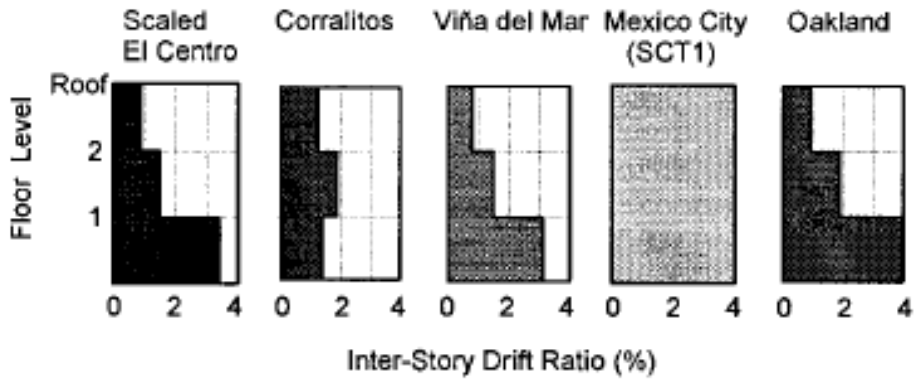


Figure 2-24 Distribution of maximum inter-story drifts for unretrofitted three-story building under the seismic records studied

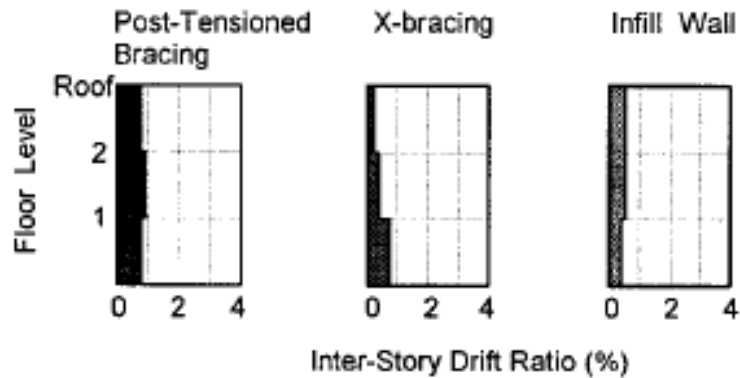


Figure 2-25 Distribution of maximum inter-story drifts for retrofitted three-story buildings under the Corralitos record

The second prototype structure was a 12-storey building retrofitted by two different bracing schemes and 2 more shear wall patterns as represented in Fig. 2-26. The same set of analyses were conducted for these building as well. The results showed different patterns of response as compared to the low-rise buildings. The maximum base shear force occurred when X braces were used. The base shear-lateral drift envelopes are compared in Fig. 2-27. The result indicated failure of the un-retrofitted building at its second floor. For the high-rise building all the retrofitted structures survived under the El-Centro seismic record but the building retrofitted by post-tensioned rebars collapsed under the Mexico City Earthquake record. The failure was observed in the third floor.

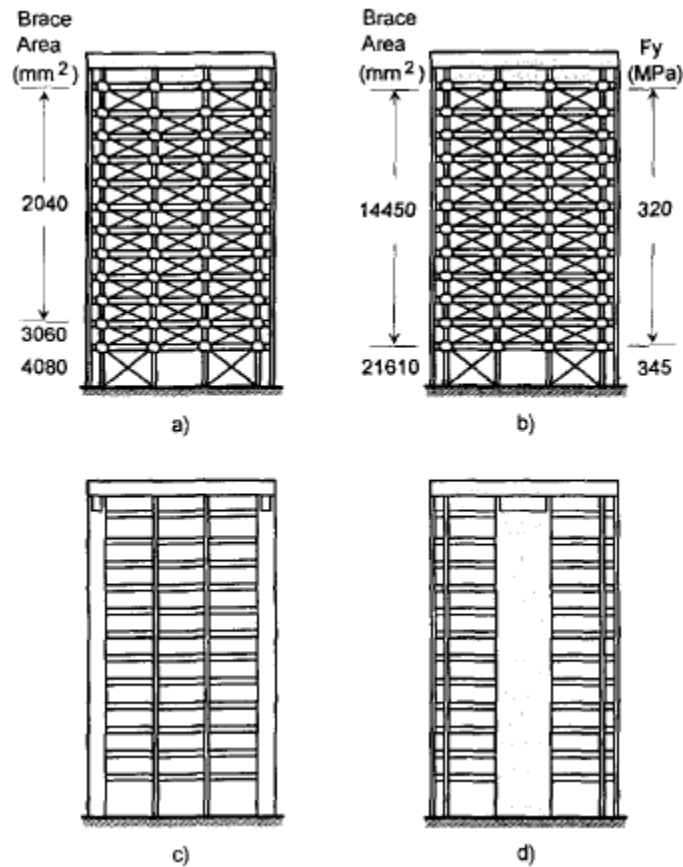


Figure 2-26 Retrofit Schemes for 12-Story Building: (a) Posttensioned Bracing; (b) X-Bracing; (c) Wall Scheme W1; (d) Wall Scheme W2

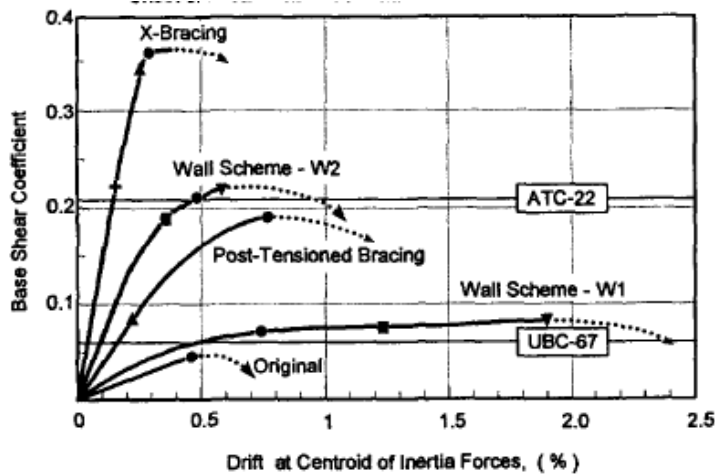


Figure 2-27 Comparison of base Shear coefficient and drift relationships for original and retrofitted 12-story building

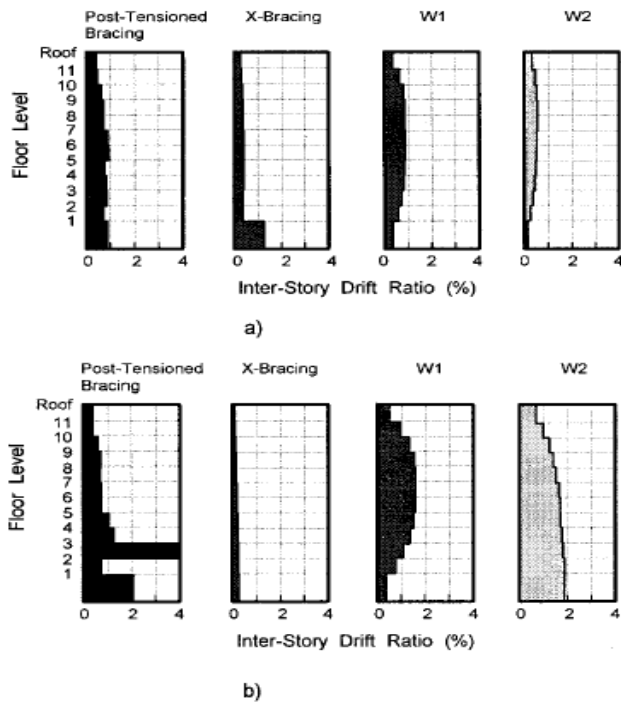


Figure 2-28 Distribution of maximum inter-story drifts for retrofitted 12-story building subjected to records from (a) EI Centro (Scaled) b) Mexico City

The third building height investigated was a mid-rise building having 7 stories. This building was retrofitted with X bracing and post tensioned braces. Both techniques provided sufficient resistance against different seismic records though their behavior was different. Fig. 2-29 shows the inter-storey drift ratios obtained from different earthquake records.

Use of prestressing strands as tension braces has been investigated at the University of Ottawa. Large-scale reinforced concrete frames were designed and built based on the 1963 ACI-318 building code to represent the practice in the pre-1970 era of seismic design. Shalouf and Saatcioglu (2006) tested concrete frames with brick infill walls that were seismically retrofitted by diagonally prestressed strands. Fig. 2-30 illustrates the test specimen. The results indicated an increase in lateral force capacity, as well as the ductility of the system. The researchers also conducted non-linear time history analysis of a 5-storey prototype building that was subjected to the design level earthquake records for Vancouver, BC. Different level of prestressing and number of strands were used to retrofit one of the bays of three-bay frames. Fig. 2-31 shows that a significant drift control can be attained by tension-only prestressing strands as bracing elements.

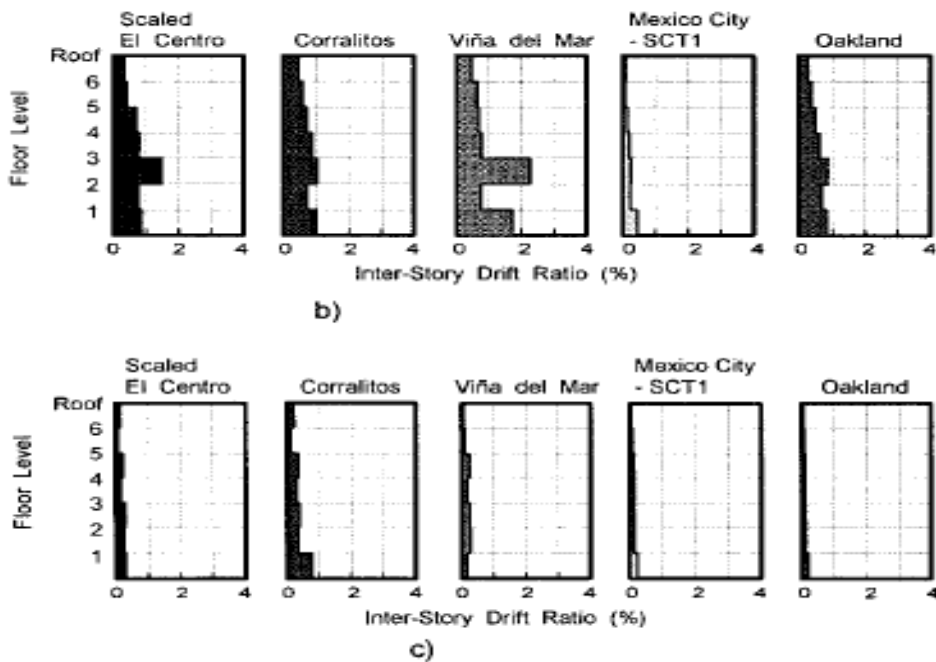
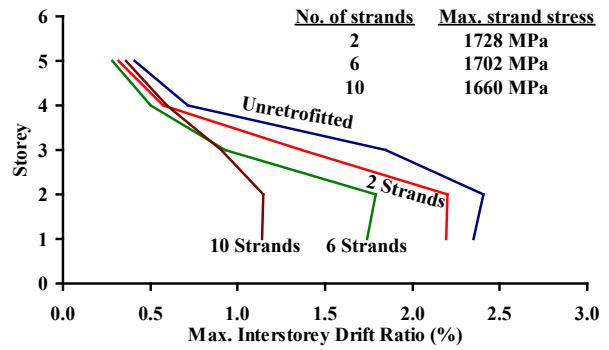


Figure 2-29 Distribution of maximum inter-story Drifts for building; (b) Posttensioned Bracing; (c) X-Bracing



(a) Test Specimen



(b) Envelop curves of dynamic response

Figure 2-30 Seismic retrofit through diagonal prestressing (Shalouf and Saatcioglu 2006)

As continuation of the above project Carrière (2008) conducted analytical investigation involving reinforced concrete frame buildings of different heights located either in Vancouver or Ottawa. The buildings consisted of 5, 10 and 15 stories. They were analysed under the 2005 National Building Code of Canada (NBCC (2005)) compatible ground motion records for Ottawa and Vancouver. Each building was analyzed twice, with and without retrofitting by diagonal prestressing without the ductile design detailing. The results indicated improvements by tension-only braces. However, the increase in the level of prestressing resulted in increased stiffness and reduced building period, attracting higher seismic forces. In a subsequent experimental phase of the same overall research program different levels of prestressing was introduced to a single bay-single storey reinforced concrete frame by Molaei and Saatcioglu (2014). The frame was designed to be seismically deficient. The clear span was 3.5 m and the clear height was 3.0 m as shown in Fig. 2-31.

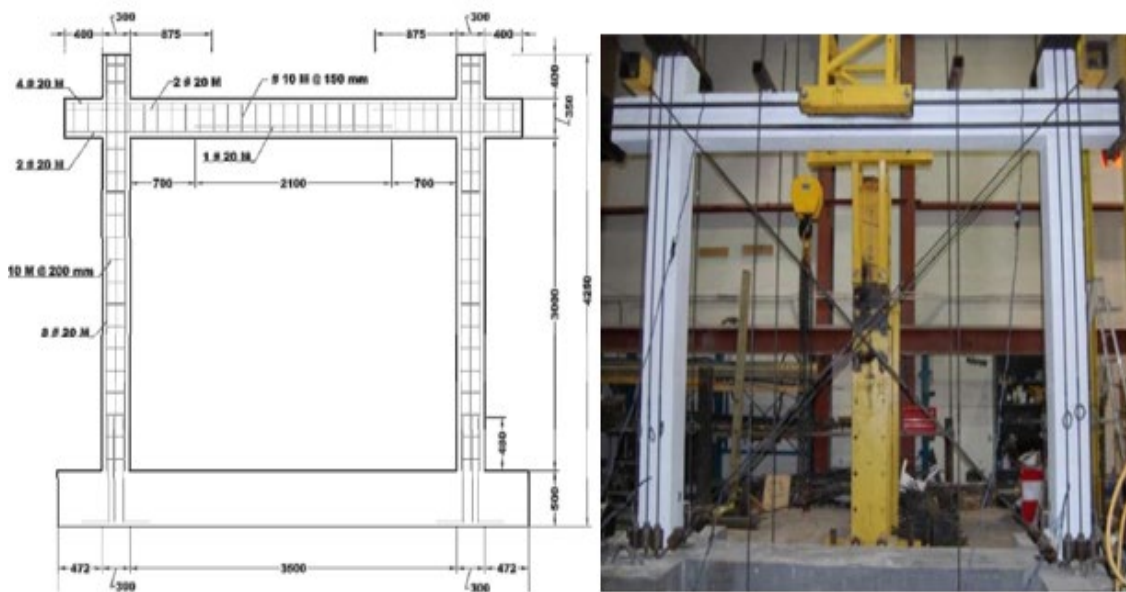


Figure 2-31 Seismic retrofit of the RC frame by pre-stressed tension-only high strength strands (A.Molaei 2014)

In one of the three cases considered, two diagonal strands were prestressed to 200 kN in each direction. The other cases involved prestressing a single strand to 100 kN and the use of two strands of the same size but without any prestressing. The results showed that the diagonal tension-only strands improved the frame lateral load capacity. When compared with the un-retrofitted frame test, the capacity of the frame increased by up to a factor of 2.0. When two prestress strands were used the failure took place at an early lateral drift ratio of 1.3% because of excessive compression force generated on the columns. By using two non-prestressed cables the capacity increased by the same amount meanwhile the ductility of the frame was 1.5 times of the case with prestressed cables. By having only one prestressed cable as the brace, the capacity increase was limited to a factor of 1.5. However, the failure drift was increased to 2%.

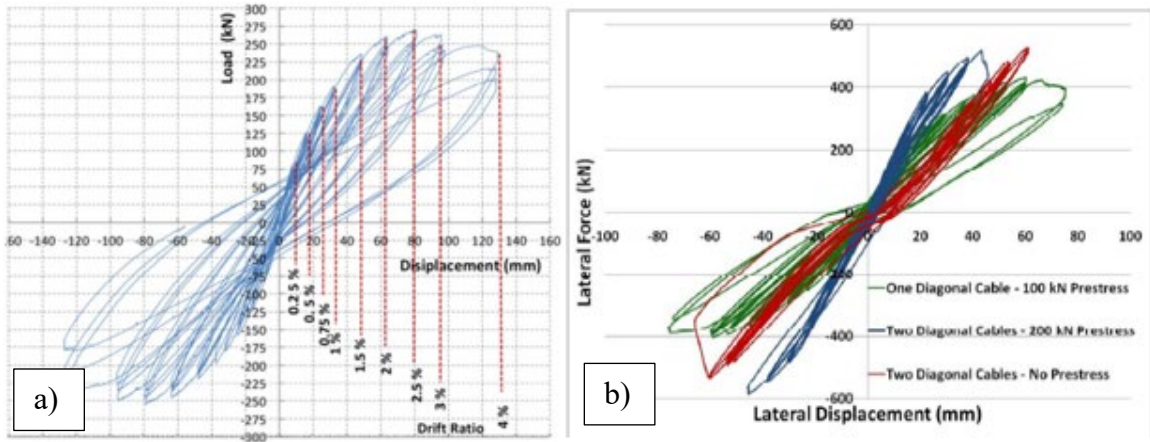


Figure 2-32 Hysteretic response of a) bare frame b) retrofitted by pre-stressed/non-prestressed strands (A.Molaei 2014)

Even if the tension-only diagonal cable system showed strength enhancements, the deformability of the frame did not improve, resulting in reduced energy dissipation. Also, a considerable sagging was observed in the cables upon load reversals due to the inelastic deformations in the steel cables. The brace yielding was observed to take place prior to reaching 2% drift, and the observed slack led to pinching of hysteresis loops.

Recently the traditional bracing systems mentioned above are replaced by new materials or more advanced innovative techniques. For example, the traditional X braces were replaced by CFCC (carbon fiber composite cable) in non-compression braces by Lee (2015) in South Korea. In this research different diameters of the CFCC were tested under tension to establish differences in behaviour between the braces of different size CFCCs. CFCC was able to reach 2% of strain under tension forces, indicating higher deformability as compared to the mild steel with higher modulus and yield strain of 0.002. A 25mm diameter cable was used to retrofit a half-scale concrete frame. Two vertical point loads applied at the top of the columns simulated the weight of the upper floors, while a hydraulic jack provided push-pull cyclic loads on the braced frame. Fig. 2-33 shows the details of the test. The results of the retrofitted frame compared to the bare frame test results showed a strength increase of a factor of 1.8. The increase in stiffness caused additional shear and compression forces on the columns. Fig. 2-34 shows the frame failure at the end of the test. Fig. 2-35 illustrates the hysteretic behaviour of unretrofitted and retrofitted frames.

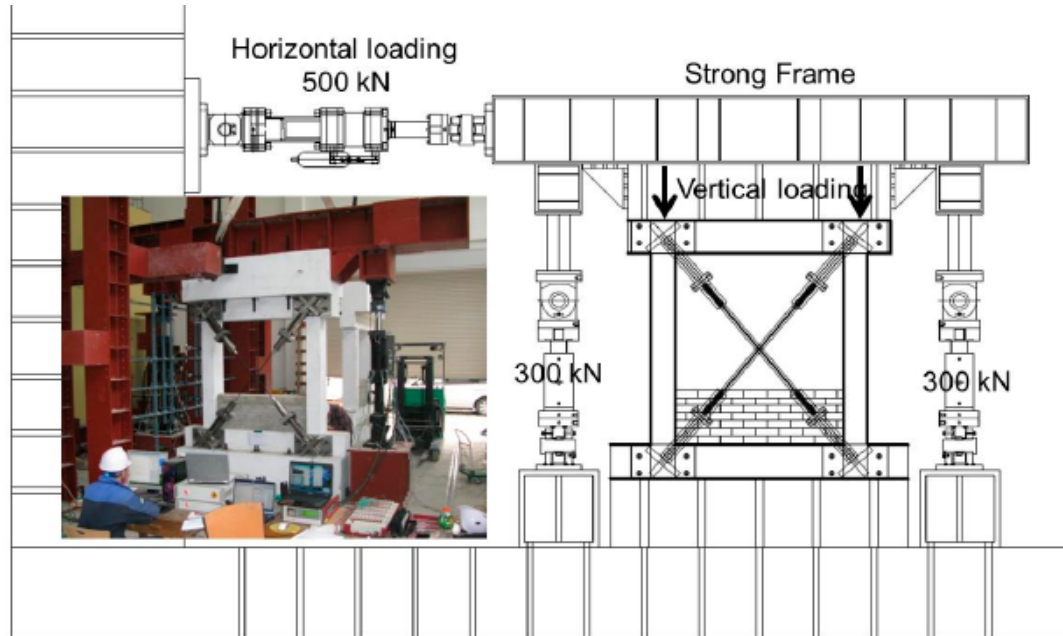


Figure 2-33 Experimental configuration of the RC frame retrofitted by CFCC (Lee 2015)



Figure 2-34 Failure of the columns at the end of the test (Lee 2015)

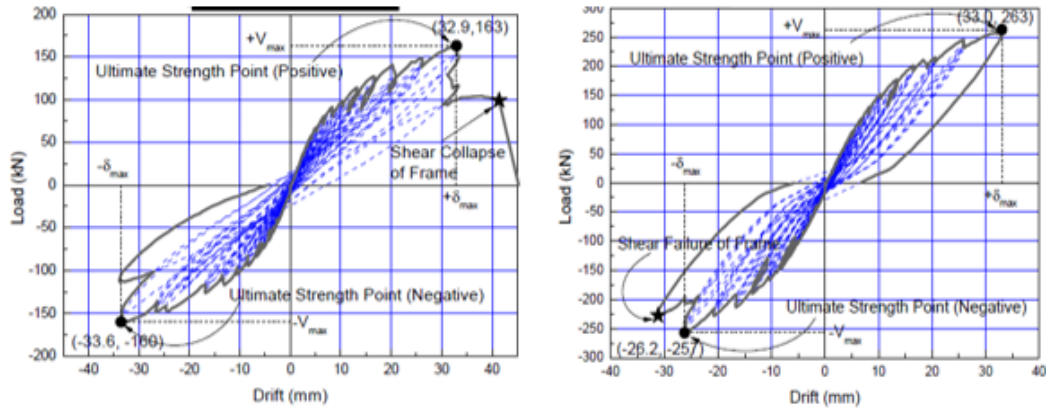


Figure 2-35 Cyclic response of un-retrofitted and retrofitted frames (Lee 2015)

Retrofitting by adding lateral stiffness to structures can cause additional forces on specific elements, which may cause local failure or structural collapse. Hou and Tagawa (2009) proposed a new steel braced frame that included the consideration of the behaviours of beams, columns and braces simultaneously. High axial forces generated by braces were incorporated in the design of the entire frame. The researchers mentioned that in regular braced frames excessive compression forces imposed on adjacent columns may cause column failures. They proposed a new seismic retrofit method to strengthen steel frames without reducing their ductility while improving seismic resistance and limiting drift within a specific range. This study incorporates a cylindrical member that bundles the wire-rope bracing elements at the intersection point as illustrated in Fig. 2-36.



Figure 2-36 Seismic retrofitted frame lab specimen retrofitted by pre-slacked braces

In this technique, the brace was not designed for small and medium vibration amplitudes. Only in large vibration amplitudes the brace acts in tension, resisting seismic forces and limiting lateral drift to code-recommended allowable limit. Thereby, the braced frames can show ductile behaviour and absorb more seismic energy, similar to the moment frames, without imposing excessive forces on the columns. They used wire cables to avoid buckling of compression braces. The proposed bracing system is designed to restrain lateral drift of a storey within a specified range. It was tested under cyclic loading on portal moment frames to reveal fundamental characteristics of the system. Three different scenarios were considered as illustrated in Fig. 2-37. The first scenario was an un-braced steel frame, the second was retrofitted the frame braced by the new bracing system and the third was a steel frame braced retrofitted by normal steel strands.

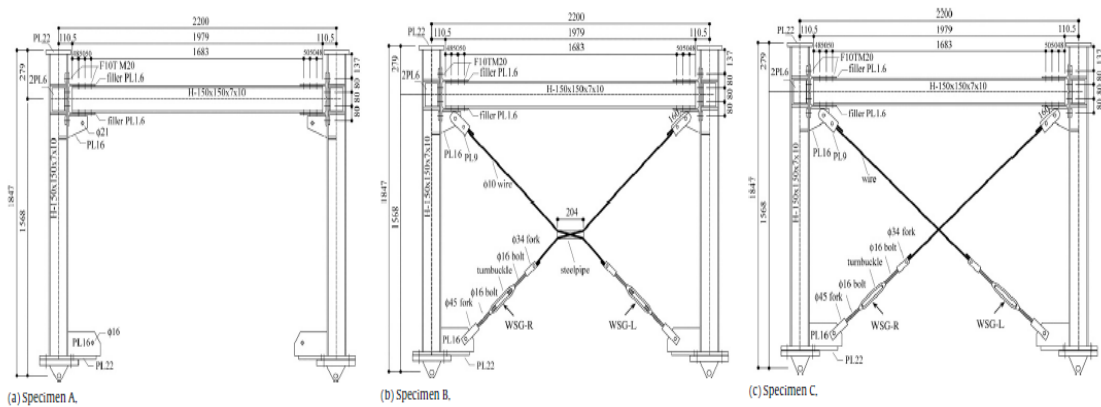


Figure 2-37 a) Unbraced moment resistant steel frames b) Moment resistant steel frame braced with pre-slacked steel strands c) Usual method of bracing steel frame with steel strands.

The specimen with the new bracing system had longer strands to accommodate the steel pipe placed in the centre overlap region. This configuration eliminated tension to be developed in the strands before the frame reached a specific drift level. The use of a pipe in the centre of the tension brace system is shown in Fig. 2-38. To investigate the effect of the stiffness of the pipe on the system, a plastic pipe was first used, followed by the use of a steel pipe.



(a)

(b)

Figure 2-38 Braces tied in a) in plastic pipe b) in steel pipe

The result for the two types of pipes are compared in Fig. 2-39 and indicate that the low-stiffness pipe resulted in the development of tension forces in strands when the target storey drift was reached. When the stiff steel pipe was used, the bracing members started becoming active earlier.

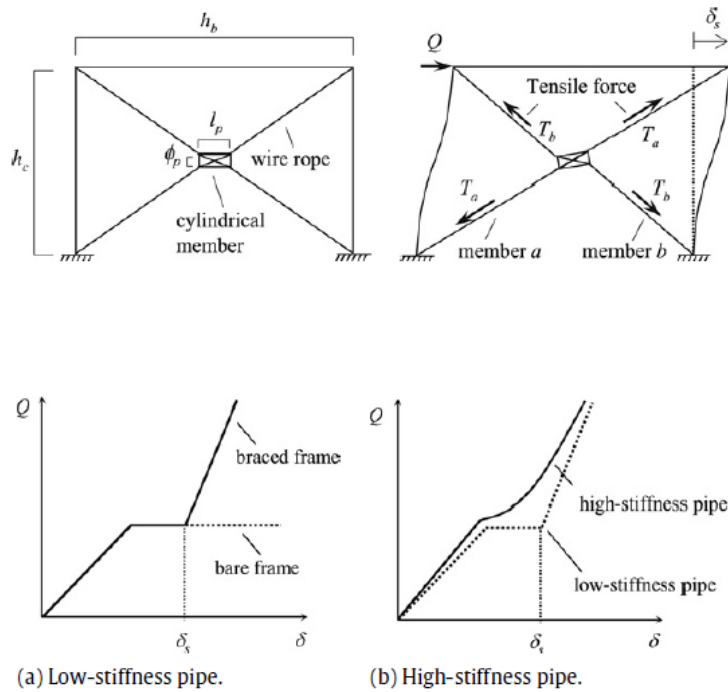


Figure 2-39 Force-displacement behaviour of the frame for braced frame a) braces tied in plastic pipe b) braces tied in steel pipe

Hysteretic force-displacement relationships obtained from the test program are shown in Fig. 2-40, illustrating the three different responses obtained from the tests. The pipe provides a slack in the middle of the braces to avoid having tension forces in small drifts. Fig. 2-41 shows the comparisons of the envelop curves for the three specimens tested.

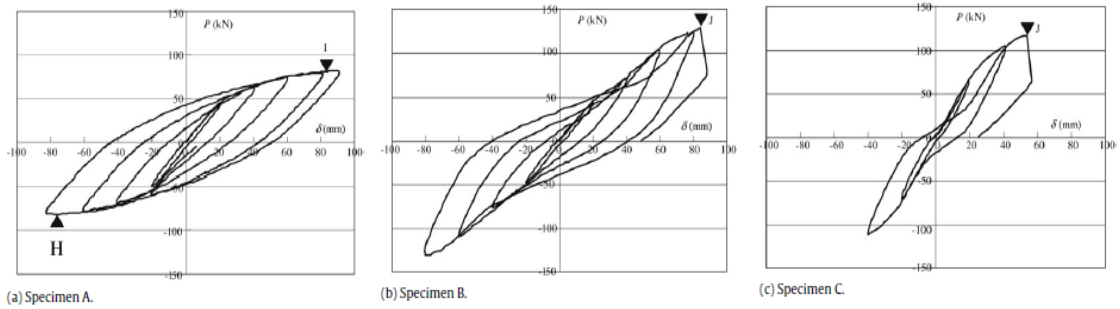


Figure 2-40 Hysteretic behaviour of the frame for a) Bare frame b) Proposed braced frame c) Typical braced frame

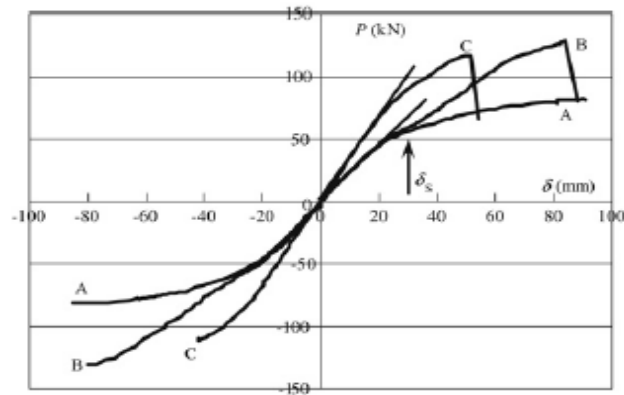


Figure 2-41 Comparison of the envelop curves for frames tested; a) Bare frame b) Proposed braced frame c) Typical braced frame

Hou and Tagawa (2009) also conducted seismic analysis of three-storey moment-resisting frames with and without bracing and showed the effectiveness of the proposed bracing system. The analyses results showed the stiffness of the frame increased after a specific drift when the brace became tightened. Having a loose brace at the beginning avoided having high base shear forces and high compression forces on adjacent columns. In addition to providing more ductility for the modelled frames the new bracing system reduced residual deformations.

Mousavi and Zahrai (2017) investigated the effects of frames braced by pre-slack strands numerically. Pre-Slacked Cable Brace (PSCB) was proposed, and the results showed improved post-yield stiffness of highly pinched non-ductile frames, with minimal increase on their lateral strength capacity. An opensource time domain platform, called Macro-Simulink model, was used for the analysis. Macro-Simulink can simulate different deteriorating hysteretic behaviours under dynamic and quasi-static excitations. In addition, the results showed no additional load on other parts of the structure compared to typical steel braces. Furthermore, self-centering specification of the braces helped frames to restore their original shape after unloading. The researchers also modelled the three frames tested by Hou and Tagawa (2009) using Macro-Simulink. The analysis results matched perfectly to those obtained in the test program.

Mousavi and Zahrai (2017) extended their analysis to include different deteriorating hysteretic behaviours, i.e., Smooth Hysteretic Model for the bare frame and Polygonal Hysteretic Model for the pre-slacked brace. Fig. 2-42 illustrates the improvements obtained in the frame with pre-slacked bracing system. Also, there is no additional stiffness during the small drift cycles.

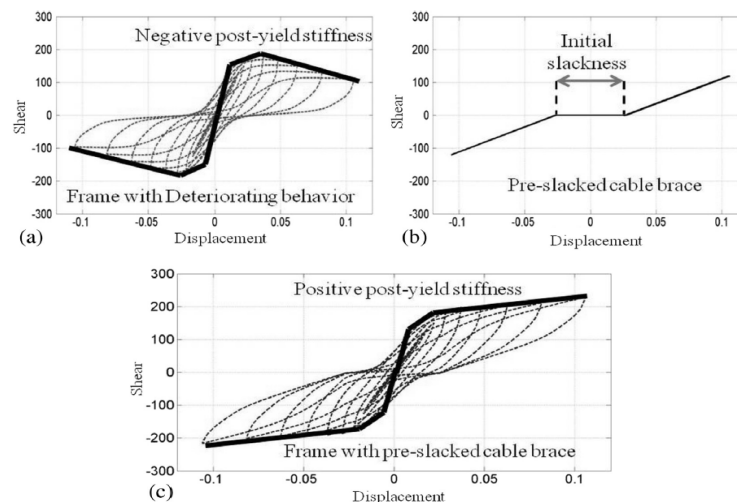


Figure 2-42 Improvement of seismic response of the non-ductile frame, retrofitted by pre-slacked braces (Mousavi and Zahrai (2017)).

At the end two configurations were shown by Mousavi and Zahrai (2017) for a frame with no opening, and a frame with a large opening. A solution, which was proposed for a

frame with openings is shown in Fig. 2-43.

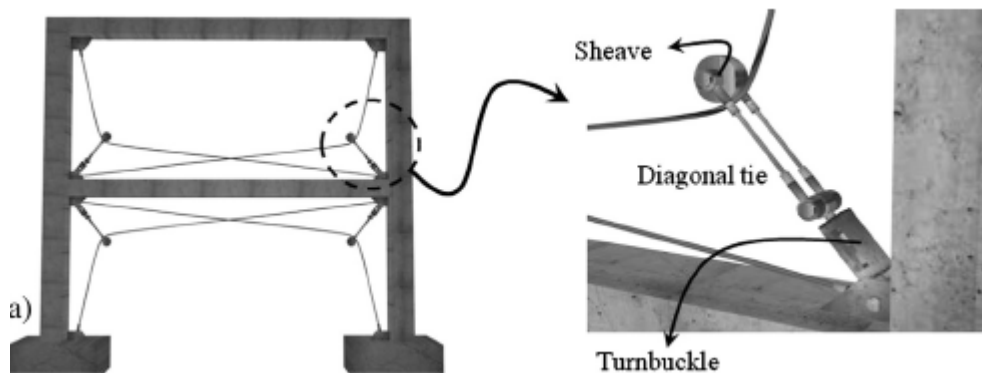


Figure 2-43 Seismic retrofitting of the multi-storey frames having openings with pre-slacked braces

Another numerical research project was conducted by Zahrai et al. (2016) involving an innovative device to remove the effect of pinching in the frames retrofitted by tension-only braces. Previous research, including tests on frames retrofitted by tension only braces or buckling restraint braced frames showed significant pinching in their hysteretic relationships, including tests conducted at the University of Ottawa by Molaei and Saatcioglu (2014). The effect of pinching was eliminated by mono-directional self-tensioning connections. The research included numerical modeling of restrained or pin frames with pre-stressed/non-prestressed tension-only X braced frames, by using Matlab/Simulink. In this analytical research, an innovative device, called “slack free connection (SFC)” was developed and modelled to show its effectiveness in eliminating the pinching effect. This device has properties of monodirectional and self-tensioning characteristics. The braces can be connected to this device by a coupler at the end of the SFC, as is shown in Fig. 2-44. When the brace is in tension, the SFC is locked, and as the brace develops tendency to generate compression force the SFC releases and avoids the slack in the system, eliminating pinching in the hysteretic relationship. In the next push, it locks the brace and transfers the tension forces through the braces.

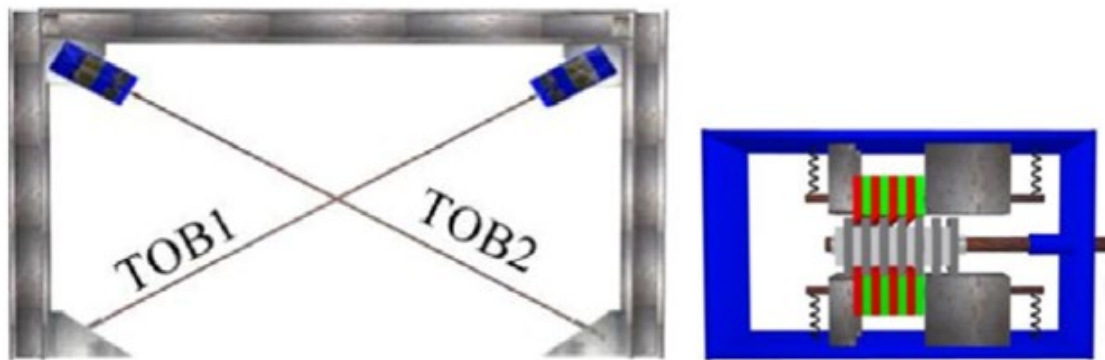


Figure 2-44 Retrofitting frame by tension-only braces equipped with SFC device

The modeling of a one storey frame retrofitted by tension-only-braces (TOBs) with/without SFC showed the advantages of using SFC devices. In all the analytical models, seismic mass of the structure was considered as 100 kg while the period was 0.63 sec. Initial lateral stiffness and the shear yield were 9950 kN/m and 196 kN, respectively. Braces consisted of mild steel with a diameter of 25mm and a yield strength of 450 MPa. For the X braced frame the lateral stiffness and yield shear force levels were close to those of the frame, but the SFC was added as part of the brace. Five analytical models were generated, Case 0 was a pinned frame alone, Case1 had plain hysteresis implemented, Case 2 had stiffness degrading hysteresis, Case 3 had stiffness and strength degradation and Case 4 had pinched stiffness and strength decay. All frames were subjected to loading regime shown in Fig. 2-45.

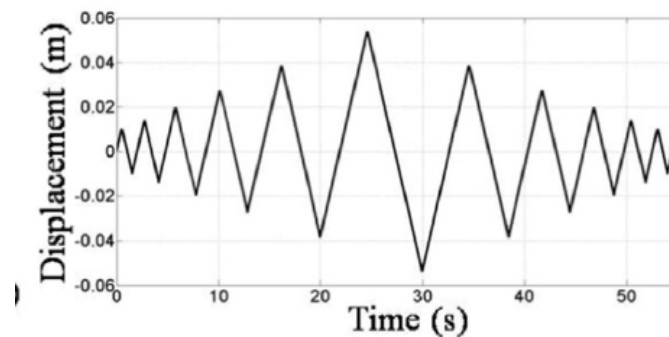


Figure 2-45 Push over time-displacement pattern for numerical analysis

Cyclic response of all the frames with and without SFC is shown in Fig. 2-46. The results showed that the use of SFC clearly eliminated the pinching effect on the frames.

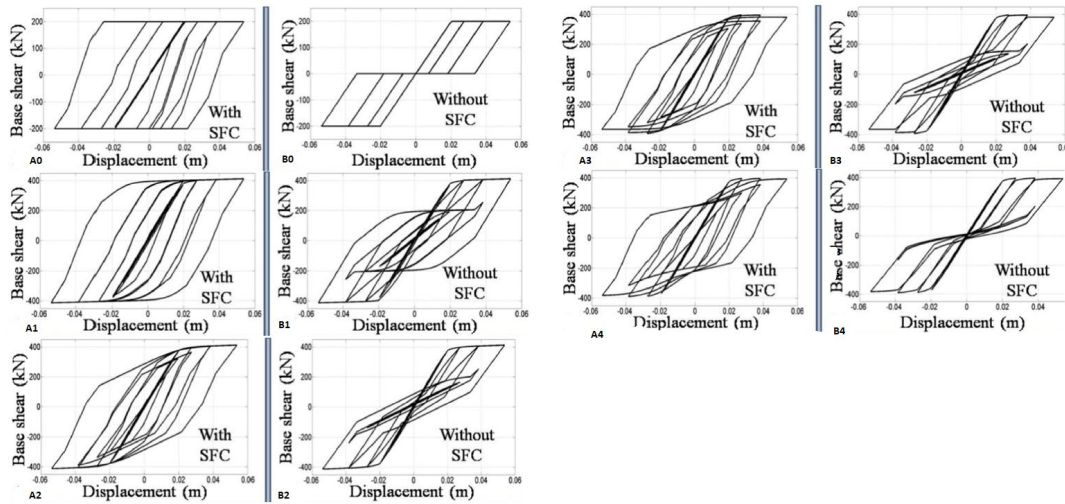


Figure 2-46 Cyclic response of the frames with and without SFC, for cases; A0 to A4 and B0 to B4

Table 2-1 summarizes the dissipated energy for each case as the area under the hysteresis loops, showing the superior performance of the frames with SFC.

Table 2-1 Energy dissipation of the frames with and without SFC

Frame hysteretic behavior	Dissipated energy (kN·m)		Increased energy dissipation due to SFC
	With SFC	Without SFC	
Case 0	89	13	585%
Case I	151	77	95%
Case II	113	39	190%
Case III	117	42	180%
Case IV	100	26	285%

2.2.3.3.2 Shape Memory Alloy (SMA) braces

Shape memory alloys are materials that can undergo large deformations under axial loads with an ability to recover their shape upon unloading or heating. Returning to their original length or shape by heating is known as the shape memory effect and returning to their original length and shape by unloading is known as the super-elastic effect. The unique property of shape memory alloys is the martensitic phase transformation between a crystallographic austenitic phase to a low-symmetry martensitic phase. At low temperatures

and high stresses, usually the martensite phase is stable. Austenite phase has more stability at lower stress with higher temperature. The SMA shows shape memory effect when the temperature is below the temperature that corresponds to the end of the martensite phase. Deformations caused by forces can be recovered by heating the material at the temperature above the austenite level. Under loading and un-loading, SMA shows martensite to austenite at a lower stress, thereby resulting in the super-elastic behaviour. Fig. 2-47 shows the characteristics of a typical NiTi SMA material.

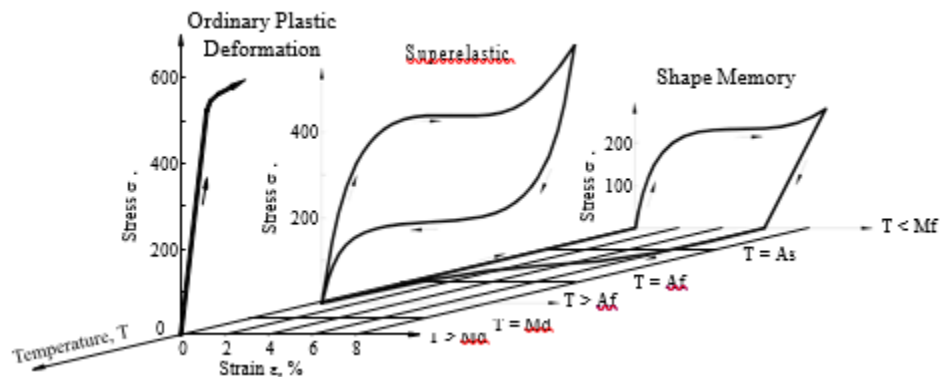


Figure 2-47 Three-dimensional stress-strain temperature diagram of Shape Memory Behavior of NiTi

Super-elastic shape memory alloys possess several characteristics that make them desirable for applications in seismic-resistant design and retrofit of structures. These characteristics include large elastic strain range, hysteretic damping, re-centering capabilities, strain hardening at large strains, and excellent low and high-cycle fatigue properties. Mechanical and chemical properties of NiTi shape memory alloys is shown in the Table 2-2.

Kawaguchi et al. (1991) studied the properties of super-elastic shape memory alloys, including their behaviour under cyclic loading and rates expected during earthquakes. In one of these studies SMA wires and bars up to 25.4 mm diameter were tested under cyclic tension loading up to 6% of their longitudinal strain to determine the mechanical properties of super-elastic shape memory alloys. The mechanical properties that were evaluated included reverse and forward transformation stresses (σ_L , and σ_{UL}), the residual strains (ϵ_r),

and energy dissipation (σ_{eq}). Residual strain is defined as the strain that remains on SMA after unloading.

Table 2-2 Mechanical properties of NiTi Shape Memory Alloys

Property	NiTi SMA	
	Austenite	Martensite
Physical Properties		
Density	6.45 g/cm ³	
Mechanical Properties		
Recoverable Elongation	up to 8%	
Young's Modulus	30-83 GPa	21-41 GPa
Yield Strength	195-690 MPa	70-140 MPa
Ultimate Tensile Strength	895-1900 MPa	
Elongation at Failure	5-50% (typically 25%)	
Poisson's ratio	0.33	
Chemical Properties		
Corrosion Performance	The same as stainless steel	

SMA's are also produced in the form of wires, such as the Nitinol SMA. DesRoches et al. (2004) investigated the cyclic loading effect on the residual strain of 1.8 mm diameter SMA wire. The results are shown in in Fig. 2-48. The initial modulus of elasticity obtained from the tests was approximately 40 GPa. During the early cycles, the forward transformation stress was observed to be around 600 MPa (87.0 ksi). The effect of fatigue on cyclic loading resulted in a decrease in σ_L to almost 400MPa (58 ksi) during the later cycles. The effect of small slippage on the cyclic behaviour can result in lower values of the forward transformation stress, σ_L (Miyazaki et al. (1986); Miyazaki and Otsuka (1989); Tobushi et al. (1992). The unloading path stress transformation (σ_{UL}) was observed at 200 MPa (29 ksi) for the 2% cycle to the first 6% cycle. Slight reductions were observed during subsequent cycles at 6% strain. Other researchers showed that under increased number of cycles, the decrease in unloading stress plateau is less than that for the loading plateau (Strnadel et al. (1995); 1995b; Friend and Morgan (1999). The test results showed that equivalent viscous damping decreased during loading cycles at 6% strain. Also, ϵ_{UL}

remained constant during the cyclic loading test while the ϵ_L decreasing, resulting in smaller hysteresis curves. Dolce and Cardone (2001) had similar studies and the results showed maximum equivalent viscous damping of at around 5.5% and the maximum strain of the wires was in the range of 5-7%, confirming earlier tests.

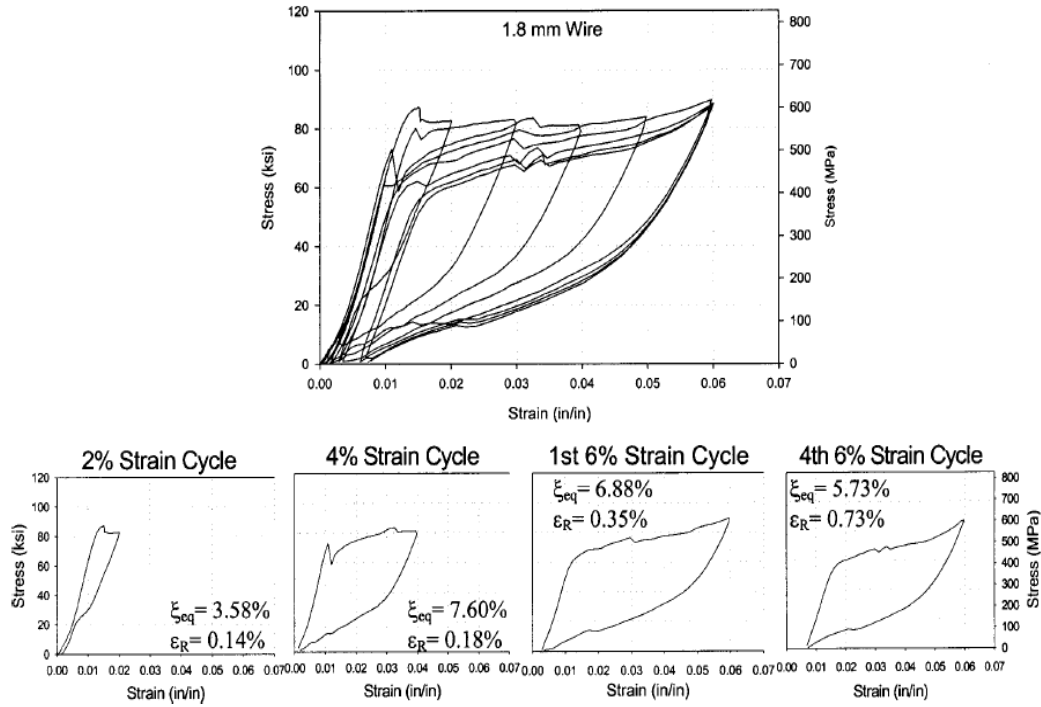


Figure 2-48 Stress-Strain Behavior of a 1.8 mm diameter Nitinol Wire Subjected to Quasi-Static Cyclic Loading.

Research conducted in the early 1990s showed that SMA bars have different properties compared to what was found for wires. Dolce et al. (2000), recommended using SMA wires rather than SMA bars because of the enhanced performance of wires. Cyclical stress-strain relationship for a 25.4 mm diameter SMA bar subjected to the same loading protocol as wire showed an initial modulus of elasticity of 28 GPa with loading plateau stress of 410 MPa, which is almost 30% below the values obtained from the same material wire. Due to the fatigue effect, σ_L observed in the test decreases to 375 MPa during the following cycles. Morgan and Friend (2001) investigated the reverse transformation stress, or unloading plateau, σ_{UL} , using thermo-mechanical processing and found that it was approximately 268 MPa during 2% strain cycles and decreases to approximately 260 MPa at 6% strain cycles.

This is shown in Fig. 2.49.

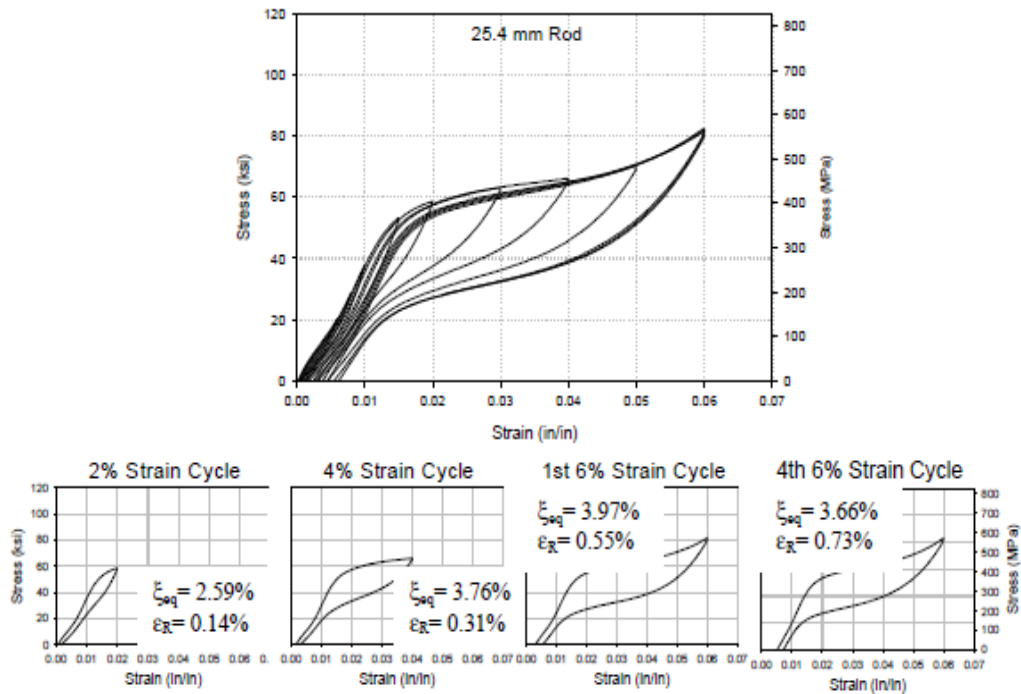


Figure 2-49 Cyclic Stress-Strain for a 25.4 mm diameter Nitinol Bar Subjected to Quasi-Static Loading.

The researchers reported that the residual strains in SMA increased after experiencing increased number of cycles. After four cyclic loading at 6% strain, there was 0.73% residual strain whereas it was 0.14% after the 2% strain cycles.

Tobushi et al. (1998), showed that as the strain rate increased above 0.16%/s, the energy dissipation increased, compared with statically loaded SMAs. However, an accurate test on rate-dependent coupled thermo-mechanical response of shape memory alloy bars and wires in tension was done by Mirzaeifar et al. (2011). The researchers investigated different size SMA wires and Rods. Also, the effect of temperature and the frequency of loading was investigated when the material was in tension. The experiment results showed a very good correlation with analytical predictions for 7.1mm diameter rods as illustrated in Fig. 2-50.

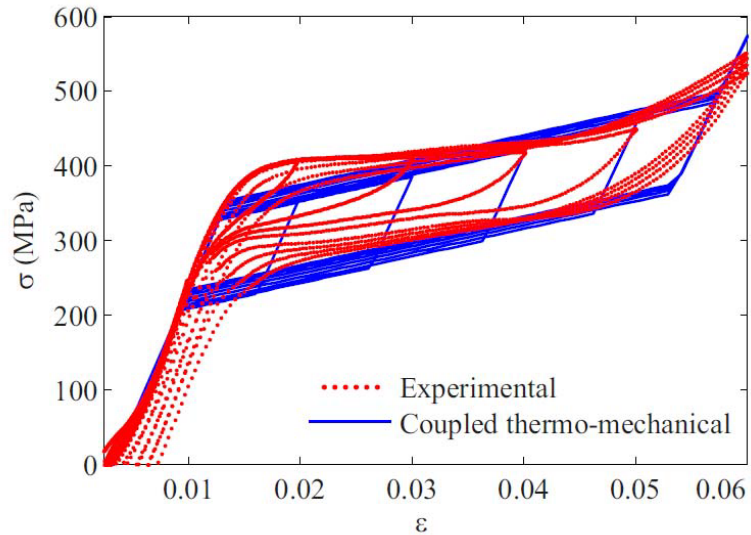


Figure 2-50 Comparison of the experimental and analytical results for the stress-strain response of an SMA bar with $d = 7.1\text{mm}$

Fig. 2-51 shows the effect of loading rate on the behavior of an SMA rod with 50mm diameter. It can be observed that the stress-strain relationship is linear up to a strain of $\epsilon = 0.01$ and the corresponding stress is nearly $\sigma = 600\text{MPa}$. Following strain hardening the unloading curves indicate a flag-shape cyclic behaviour.

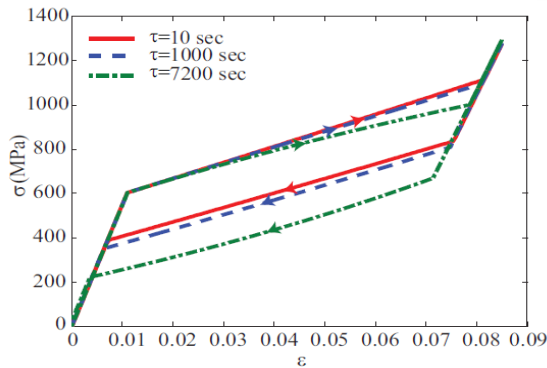


Figure 2-51 The effect of total loading-unloading time τ on stress-strain response at the center of an SMA bar with $d = 50\text{mm}$

The cyclic stress-strain responses of both SMA wires and rods were found to be similar in terms of loading, unloading and energy dissipation, since the samples were heat treated and tested under the same conditions and temperature. Both cases showed perfect flag shape hysteretic behaviour and a small residual strain at the end of unloading which makes this

material suitable for seismic retrofits with self-centering features. The only difference between the two appears to be the higher loading stress and lower unloading stress in wires, which can lead to a higher equivalent viscous damping relative to bars. A negative aspect of using SMAs is their sensitivity to their material composition and thermo-mechanical heat treatment. For example, 10% more Titanium in NiTi causes a significant effect on the loading plateau stress and the hysteretic stress strain path (Serneels (1999)). The production of these materials needs a special accuracy. In terms of the cost for producing, it is more expensive to produce large diameter rods than small diameter wires, But the price of the production has come down in recent years with increased usage of large diameter rods in industry.

In 1991, Graesser and Cozzarelli (1991) suggested using nitinol (nickle titanium), which is a type of shape memory alloy to be used in seismic resistant design of structures. They suggested using nitinol as dampers in structures. Their studies focused on loading frequency and history on the energy dissipation characteristics of nitinol wires. Based on their research, they concluded that nitinol wires could be modelled as one-dimensional constitutive models having superelastic behaviour as verified by experimental work. Inaudi and Kelly (1994) modelled a four-story steel-frame with tuned mass dampers made of shape memory alloy wires. They changed the number of SMA wires the prestressing load to observe the seismic response of the building. The results showed significant improvement of building dynamic response when the prestress tension was tuned to the first natural frequency of the isolated structure.

Krumme et al. (1995) and Clark et al. (1995) investigated the application of SMA in building seismic design. They concluded that SMA wires could reduce lateral drift and response accelerations, leading to reduced base shear. Seismic research involving the use of SMA dampers on bridges was conducted in Japan (Adachi and Unjoh (1999); Wilde et al. (2000)). They concluded that the use of SMA wire dampers reduced lateral displacement of the bridge deck. Ohi K. (2001) tested the use of NiTiCo SMA braces and found out that the structure could go back to zero deflection after reaching 5% of strain in tension. In addition, the brace provided some moderate damping for strains higher than 1%.

Research on use of SMA in buildings increased in recent years. Ocel et al. (2004) tested beam-column connections using 35 mm diameter SMA rods. These connections were stable under cyclic loading, up to 4% of storey drift. They investigated the effect of heating the connection after the first test. It was reported that heating recovered approximately 76% of residual deformation.

European Union started a multi-year research program in late 1990s named MANSIDE to investigate the use of SMA in seismic isolators and energy dissipation devices. Analytical modelling and experimental testing of SMA wires and SMA bars were considered (Dolce et al. (2000); Dolce and Cardone (2001)a, (2001)b). Their results suggested that super-elastic properties are difficult to achieve for large diameter SMA bars.

Cortés-Puentes and Palermo (2018) used 12.7 mm SMA bars in tension braces for seismic retrofit of reinforced concrete squat shear walls. The shear walls were designed based on the 1965 edition of the National Building Code of Canada for western Canadian seismicity. A typical test specimen is shown in Fig. 2-52. The braces consisted of HSS sections with SMA bars in the middle. Three walls were tested; two retrofitted with SMA braces and one un-retrofitted as a control specimen. Fig. 2-53 shows the comparison of force-deformation relationships for retrofitted and un-retrofitted walls.

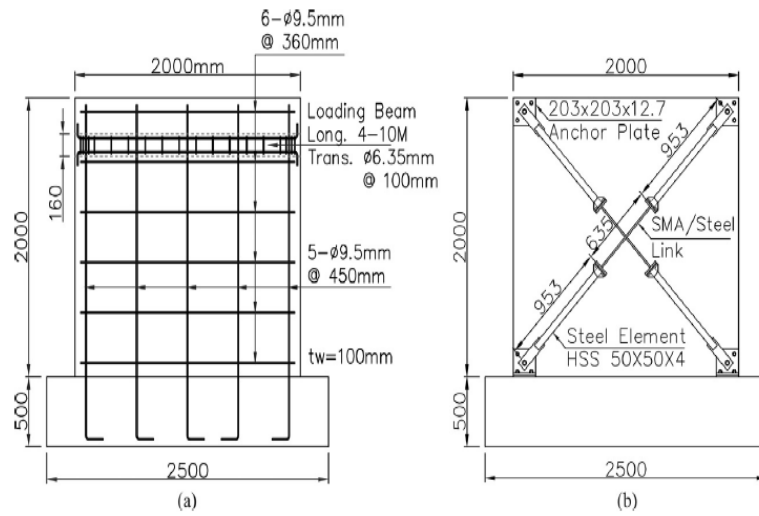


Figure 2-52 Non-ductile squat wall retrofitted by HSS and SMA braces (Cortés-Puentes and Palermo 2018)

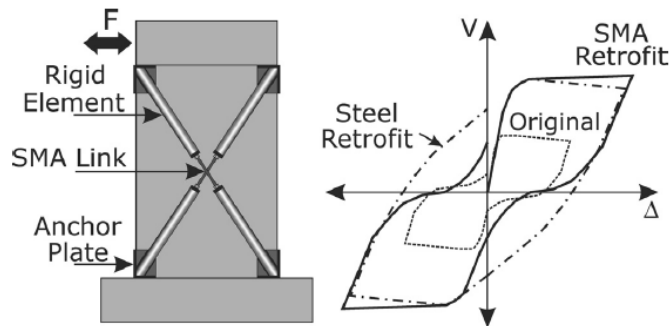


Figure 2-53 Comparisons of wall behaviour with and without lateral braces (Cortés-Puentes and Palermo (2018)

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CHAPTER 3

3 Seismic Retrofit of Non-Ductile Concrete Frames with Cable-SMA Assembly

Abstract:

Reinforced concrete buildings built prior to the enactment of modern seismic codes are often seismically deficient. These buildings lack sufficient strength and ductility to withstand strong earthquakes. Conventional retrofit techniques for such frame buildings involve adding reinforced concrete shear walls or structural bracing systems to the existing bays of frames. These techniques are often intrusive and result in lengthy building down periods and expensive structural interventions. An alternative to conventional techniques is the use of high-strength prestressing strands or cables, diagonally placed as tension elements. This technique was researched and used in a limited manner after the 1985 Mexico City Earthquake. One of the unavoidable features of the tension bracing systems is to increase building rigidity, thereby reducing building period and increasing seismic force demands, sometimes leading to further strengthening requirements in adjacent structural elements and their foundations. Furthermore, upon yielding, cable braces develop permanent plastic deformations, resulting in excessive pinching of global hysteretic response, reducing energy dissipation and resulting in permanent building sway after the earthquake. While the use of tension cables may be desirable for drift control, it may result in undesirable structural performance under strong earthquakes. The use of shape memory alloys (SMA) with flag-shape hysteretic relationships, placed in the middle of tension cables, provides potential improvements to the observed deficiencies of bracing systems while promoting self-centering capabilities. The use of such a system is investigated in the current paper. The cable-SMA assembly was implemented in selected older concrete frame buildings in Vancouver as a region of strong seismicity in western Canada and analyzed under code-compatible earthquake records. The buildings consisted of a 2-storey low-rise residential building and a 6-storey mid-rise office building, designed based on the 1965 edition of the National Building Code of Canada (NBCC) as representatives of pre-1970s older building designs. Inelastic response time history analyses were conducted to assess the performance of cable-SMA retrofit system relative to un-retrofitted buildings, buildings retrofitted with high-strength prestressing strands and mild steel cables without the SMA.

The results indicate excessive storey drifts in un-retrofitted buildings, drift demands exceeding 5% and 3.5 % in mid-rise and low-rise buildings, respectively, indicating poor behaviour of conventional buildings under strong seismic excitations. Companion buildings retrofitted with the cable-SMA systems developed reduced drift demands with maximum inter-storey drifts limited to 2.5% with little or no permanent deformations. In contrast, buildings retrofitted with tension cables without the SMA resulted in high force demands, increasing base shear, inducing excessive tension and compression forces in the attached columns.

Keywords: Concrete frames, bracing, cables, ductility, drift control, earthquake engineering, shape memory alloys (SMA), seismic retrofit.

3.1 Introduction

Existing reinforced concrete frame buildings built using older editions of building codes may pose a significant risk when subjected to strong earthquakes. Seismic design and detailing requirements in building codes have improved over the years. The seismic design base shear in the National Building Code (NBCC) of Canada has changed since the inception of seismic provisions in the 1941 NBC (NRCC (1941)). Earlier design equations for equivalent static load analysis defined seismic base shear as a percentage of seismic weight of buildings. The buildings were essentially designed for gravity and wind loads. In the 1953 NBC (NRCC (1953)), the building height was introduced as a design parameter, crudely reflecting the effect of building period on seismic base shear. The differences in construction type and associated level of ductility were introduced to the code in 1965, reducing base shear for reinforced concrete frame buildings that have seismic detailing for ductile response. In the 1970 NBC (NRCC (1970)) the effect of construction type was treated more extensively through a coefficient, reflecting the associated level of ductility. Empirical expressions were introduced for the computation of fundamental period. This was followed by the 1975 NBC Commentary (NRCC (1975)) with ductility factors for different building types for use in dynamic analysis. The requirements remained essentially the same in the 1980 NBC (NRCC (1980)) with refinements made to seismic response coefficient as affected by the fundamental period. In 1990, force modification factor R was introduced to

account for the available ductility in the system, with a calibration factor U , which introduced a reduction in base shear to account for structural over-strength, while bringing the force level to the same level of safety implied in the earlier codes. Significant changes were introduced in the 2005 (NBCC (2005)) with new site-specific uniform hazard spectra having 2% in 50-year probability of exceedance. The approach was kept the same in the 2010 NBC (NBCC (2010)) with new hazard values introduced in the 2015 NBC (NBCC (2015)). Current seismic design requirements of the NBC of Canada are significantly more comprehensive and stringent than those of the pre-1970's era. In Canada, seismic design force requirements for high seismic regions have increased by as much as 100% since the early 1970's (Cheung et al. (1999)). Ductile design and detailing requirements prescribed in newer codes to reduce seismic vulnerabilities were not implemented in the majority of existing older buildings. These vulnerabilities include lack of one or more of strength, stiffness, and ductility of the lateral force resisting system. Therefore, a large number of existing buildings are seismically deficient, posing a significant threat to life safety and economy. Similar seismic vulnerabilities exist other parts of the world. As a result, there has been an increasing interest over the last four decades to implement seismic retrofits to mitigate earthquake risks in older substandard buildings.

Seismic retrofit techniques may be implemented at the local level to improve strength and ductility of individual elements or at the global level by increasing strength and stiffness of the entire seismic force resisting system, thereby controlling lateral drift and associated damage. Local retrofits may be more suitable when the number of critical elements is small, and their retrofits are sufficient to enhance the performance of the entire building. Soft-storey or weak-storey columns, short columns and shear-deficient members fall into this category. Often, however, the entire structure may be deficient in terms of seismic force resistance and inelastic deformation capacity to dissipate seismic induced energy. In such cases adding new bracing elements to resist higher seismic forces, while controlling lateral drift and limiting inelasticity in other non-ductile members may be an appropriate retrofit strategy.

Conventional seismic bracing systems include the addition of shear walls in existing bays

of frames at appropriate locations, preferably in a symmetric configuration. Shear walls can increase lateral stiffness and strength, as they attract a large portion of seismic inertia forces. However, adding new shear walls to existing buildings has its challenges. Architectural and functional requirements may prohibit the addition of shear walls at desired locations. The parts of the foundation located under these additional walls require strengthening, especially against uplift. Also, proper connections of walls to the existing structural elements, including the diaphragms should be provided to transfer seismic forces. Furthermore, the addition of new walls may be an intrusive process, increasing down time of the building. The addition of concrete structural walls has been researched extensively in the past and was shown to be an effective seismic retrofit methodology (e.g. (Altin et al. 1992); Arslan et al. (2010); Canbay et al. (2003); Frosch et al. (1996)).

Buildings can also be retrofitted using structural steel as diagonal braces. These diagonal members form tension ties and compression struts in the process of resisting lateral forces. Braces lead to increased base shear, as well as high axial compression and tension in the attached columns. They may be designed to remain elastic or may be allowed to develop plastic zones to take advantage of their strain hardening characteristics while dissipating additional seismic energy. Inelastic response of steel braces may lead to permanent deformations, sometimes developing buckling in compression. These permanent deformations can lead to permanent storey drifts, which may result in excessive pinching of global hysteresis loops. The behavior of diagonal tension and compression braces was investigated extensively in the past. Maheri and Sahebi (1997), Youssef et al. (2007), and Maheri and Ghaffarzadeh (2008) conducted experimental research to investigate the effectiveness of steel X braces in reinforced concrete frames. Ozelik et al. (2012) investigated the effectiveness of chevron braces in non-ductile reinforced concrete frames through experimental research. Other researchers also conducted experimental and analytical research to show the effectiveness of steel braces in reinforced concrete frames (Badoux and Jirsa (1990); Bartera and Giacchetti (2004); Bush et al. (1991); Bush et al. (1991); Fukuyama and Sugano (2000); Ghobarah and Abou-Elfath (2001); Kawamata and Ohnuma (1981); Maheri and Hadjipour (2003); Maheri and Sahebi (1997); Masri and Goel (1996); Mazzolani (2008); Ozelik and Binici (2008); Ozelik et al. (2011); Sugano et al.

(1980, 1981, 1983); Sugano and Fujimura (1980); Tagawa et al. (1992); Yamamoto and Umemura (1992)).

The Buckling-Restrained Brace (BRB) is another type of bracing system which can be considered as a special class of concentrically braced frames providing resistance in tension and compression. These types of braces were initially investigated and developed by Watanabe et al. (1988) in Japan. After the 1995 Kobe Earthquake, the researchers in Japan developed BRBs as a damper bracing system for seismic retrofitting. Design of typical BRBs is well explained in the literature (Tremblay et al. 1999, 2006, 2011; Uang and Nakashima (2004); and López et al. (2004)). The first usage of BRB in the United States was in 2000 after many large-scale tests conducted at the University of California by Clark (2000). Subsequently, this method of retrofit was employed in more than 350 buildings in the USA (Kimberley and Cameron (2011)). The application of BRB systems include bracing both steel and concrete frame structures, although the majority of the applications involved steel frames. One of the first research projects involving the application of BRB to non-ductile concrete frames was conducted by Al-Sadoon et al. (2020). The researchers developed a new BRB system and showed its effectiveness in concrete frame buildings.

In addition to the traditional steel braces, tension-only cables and pre-stressing strands have been used, often placed diagonally in frame bays to provide tension-only bracing against seismic forces. Pincheira José and Jirsa James (1995) conducted analytical investigations on use of tension cables for retrofitting non-ductile RC frames, focusing on a three-storey, a twelve-storey and a seven-storey building located in high seismic zones in the United States. They investigated the behaviour of buildings retrofitted by different patterns of post-tensioned braces and X braces compared to the same buildings retrofitted by infill structural walls. This was followed by an extensive research program at the University of Ottawa, consisting of large-scale frame tests and inelastic time history analysis of reinforced concrete frames to assess the significance of prestressing diagonal tension elements (Shalouf and Saatcioglu (2006), Carrière (2008), Molaei and Saatcioglu (2014)). The researchers concluded that, while the steel cable braces provided effective drift control and strengthening against seismic forces, inelastic deformations that developed in tension

ties under high levels of seismic forces generated excessive pinching of hysteresis loops, limiting energy dissipation capacity. Zahrai et al. (2016), Zahrai and Mousavi (2016), Mousavi and Zahrai (2017) developed slack free connections to overcome problems associated with plastic deformations in tension ties. The use of pre-slacked cable braces to controlling tension and compression forces in the attached steel frames was also researched by Hou and Tagawa (2009). The researchers used loose cables as diagonal bracing elements, connected together at the crossing points by a steel cylindrical element that would delay the development of tension in the cables, thereby controlling the resulting tension and compression forces in the attached steel frame elements.

The use of shape memory alloys (SMA) with flag-shaped hysteresis loops in earthquake resistant structures have recently been investigated by numerous researchers. The SMA can go through its plastic axial capacity to dissipate considerable energy without a permanent elongation after cyclic loading. This material also has self-centering properties, which make it a potential candidate for use in tension braces. Krumme et al. (1995) and Clark et al. (1995) investigated the application of SMA in building seismic design. They concluded that SMA wires could reduce lateral drift and response accelerations, leading to reduced base shear when used as dampers. Ohi K. (2001) tested the use of SMA braces in steel frames and reported that the structure could go back to zero deflection after reaching 5% of strain in tension. The European Union started a multi-year research program in late 1990s named MANSIDE to investigate the use of SMA in seismic isolators and energy dissipation devices. Analytical modelling and experimental testing of SMA wires and SMA bars were considered by others (Dolce et al. (2000); Dolce and Cardone (2001)a and 2001b). The results suggested that super-elastic properties are difficult to achieve for large diameter SMA bars. Cortés-Puentes and Palermo (2018) used 12.7 mm SMA bars in tension braces for seismic retrofit of reinforced concrete squat shear walls.

The current project is intended to demonstrate the effectiveness of using steel cable-SMA assembly as tension only braces to seismically deficient reinforced concrete frame buildings in Canada.

3.2 Research Significance

The review of previous research on seismic retrofit of non-ductile reinforced concrete frame buildings indicates that new seismic retrofit techniques with superior structural performance as well as ease and speed of construction are needed for enhanced seismic risk mitigation strategies for existing non-ductile frames. The current investigation has the objective of demonstrating the effectiveness of steel-cable-SMA assemblies as a viable technique, fulfilling these requirements. The scope involves a comparative performance assessment of buildings retrofitted with three types of cable braces consisting of i) high-strength prestressing strands, ii) mild steel cables, and iii) high-strength-strand-SMA assemblies. A 6-storey office building and a 2-storey residential building, designed based on the 1965 NBC (NRCC (1965)), were used as representatives of pre-1970s design practice in Canada. The reference buildings were designed for Vancouver, a region of high seismicity in western Canada. The buildings were analyzed using dynamic inelastic response time history analysis under code-compatible earthquake records. The results are compared with un-retrofitted buildings to illustrate the effectiveness of tension cable braces and the superior performance of the steel-cable-SMA system for use in practice as a viable seismic risk mitigation technique.

3.3 Building design and dynamic analysis

Two reinforced concrete frame buildings, representing pre-1970s construction practices in Canada were designed by using the 1965 NBC (NRCC (1965)) with 6-storey and 2-storey building heights. The design internal forces were governed through the analysis of the frames carried out by using ETABS software ((CSI 2015)), and the frames were designed according to ACI (1963). The floor plan was selected as a modified version of the frame building designed in the Canadian Concrete Design Handbook (CAC 2016). Fig. 3-1 illustrates the floor plan and the 3-dimensional computer models used for design and analysis. The buildings were designed using gravity loads that consisted of dead and live loads, and lateral loads that consisted of wind and seismic loads. The gravity loads included superimposed dead loads of 1 kPa for partitions, 1 kPa for roofing and mechanical, and 1.6 kPa mechanical in the middle 5.7 m span. The design live loads included 4.9 kPa for the

first floor and 2.4 kPa for others with 1.3 kPa snow load on the roof, 25% of which was included in the seismic mass. The wind load was selected as 1.05 kPa and seismic forces were calculated as per the 1965 NBC (NRCC (1965)). Concrete compressive strength of $f'_c = 30$ MPa, and reinforcement yield strength of $f_y = 400$ MPa were used in both building designs. This resulted in 450 mm square exterior columns and 500 mm square interior columns with 400 mm x 600 mm beams for the 6-storey building; and 400 mm square columns and 400 mm x 500 mm beams for the 2-storey building. The periods of the 6-storey and 2-storey buildings were 1.65 sec and 0.84 sec, respectively, based on the eigen value analysis and cracked section stiffnesses (60% and 40% of gross section properties for columns and beams).

The buildings were analyzed using “SeismoStruct (2016)” software, which is a powerful finite element tool for non-linear dynamic response history analysis. The software has element models for beams and columns with capabilities for geometric and material nonlinearities. The flexural behaviour of beam and column sections is obtained through the integration of stresses and strains in individual fibers with nonlinear uniaxial material models assigned to each fiber. The member response is obtained by fully accounting for the spread of inelasticity along the member length by integrating individual sectional responses. This type of modelling is considered to be an accurate representation for flexural behaviour since it can consider inelastic behaviour along the entire length of a structural member. It is recommended to have at least 200 fibers in each cross section (SeismoStruct (2016)). In addition to the section fibers, up to 7 integration sections may be needed to accurately model the hardening response of members; 4 or 5 integration sections being recommended by Seismostruct Manual to assure that the element reaches its softening response range. Seismostruct also considers plastic-hinging within a fixed length of the element (Scott and Fenves (2006)). The modelling technique used requires the specification of the potential plastic hinge length. The plastic hinge length was calculated according to Paulay and Priestley (1992) formulation. The buildings and the materials were modelled as described in the next section.

The frame deformations are calculated considering inelastic behaviour of elements. P-

Delta effects are incorporated into the analysis by calculating large displacements through the total chord rotational formulation developed and implemented by Correia and Virtuoso (2006). The chord rotation formulation is based on an exact description of the kinematic transformations associated with large displacements and three-dimensional rotations of beam-column members. This leads to the correct definition of element deformations and forces, as well as to the effects of geometrical non-linearities on the stiffness matrix. The implementation of this formulation considers small deformations relative to the element's chord, notwithstanding the presence of large nodal displacements and rotations. In the local chord system of the beam-column element, six basic displacement degrees-of-freedom and corresponding element internal forces are defined, as shown in Fig. 3-2. The internal forces developed by inelastic frame elements are computed by the software. These values are correlated with axial forces, shear forces and moments (caused by flexural or torsional) at the end of each node. By finding both forces and deformations (rotations) at the ends of each element, the hysteretic behaviour can be obtained in the form of a force-displacement relationship.

Dynamic analysis was conducted using 12 different seismic records generated for western Canada (Atkinson and Macias (2009)). The records consisted of 4 short records (S1-S4) with a maximum PGA of 0.44g, 4 long records (L1-L4) with a maximum PGA equal to 0.4g, and 4 Cascadia records (C1-C4) with a maximum PGA of 0.25g. The time histories of the records are shown in Fig. 3-3. The comparisons of response spectra with the design spectrum by using "SeismoMatch" program (Seismosoft 2016) are shown in Fig. 3-4. The damping considered in all the analyses was 5% of critical damping

3.3.1 Un-retrofitted buildings

Fig. 3-5 illustrates the analytical models for the 6-storey and 2-storey un-retrofitted buildings. The floor slabs were considered as rigid diaphragms and the columns were considered to be fully restrained at the foundation level against all rotations and displacements in all directions. All the structural members were modelled using inelastic material models for concrete and reinforcing steel. The concrete model used was developed by Madas (1993) following the stress strain relationship for confined concrete proposed by

Mander et al. (1988). The cyclic behaviour of concrete, as proposed by Martínez-Rueda and Elnashai (1997) and illustrated in Fig. 3-6(a), was adopted for all frame elements. The reinforcing steel was modelled using a uniaxial stress-strain model for mild steel. It is based on the stress-strain relationship proposed by Menegotto (1973), Menegotto and Pinto (1973), followed by the isotropic hardening rules proposed by Filippou et al. (1983). The cyclic seismic loading rules were recommended by Fragiadakis et al. (2008). The model is illustrated in Fig. 3-6(b). It was indicated to be suitable for modeling the behaviour of reinforced concrete elements under reversed cyclic loading, especially for buildings subjected to seismic loading where significant load reversals might occur.

The un-retrofitted frame buildings were subjected to the 12 different seismic records indicated in the preceding section. The results indicate that the 6-storey un-retrofitted building exceeded the code specified storey drift limit of 2.5% (NBCC (2015)) and reached collapse by exceeding the crushing limit of concrete and becoming unstable after the 5% lateral drift level under the Cascadia Seismic Record, CW3. This is consistent with the response spectra given in Fig. 3-4, in which the spectral values for the 6-storey building with initial period of 1.65 sec. show the highest response accelerations for Record CW3. The corresponding maximum base shear force for the building was 7,226 kN. The first yield in the longitudinal column reinforcement occurred at 0.8% of the first-storey drift at 5,150 kN of base shear force. The crushing of cover concrete occurred at 1.33% lateral drift and the crushing of column core took place at 2.5% storey drift. The 2-storey building also developed highest response under the same earthquake record (Cascadia – CW3) with a maximum inter-storey drift of 3.69% and a maximum base shear of 3,029 kN. These response quantities are excessive for non-ductile older buildings and indicate high level of damage. Table 3-1 provides a summary of analysis results for both mid-rise and low-rise un-retrofitted buildings. Fig. 3-7 shows the first-storey base shear versus lateral drift hysteretic relationships. Also shown are the hysteretic behaviour of the critical first storey column next to a tension brace, which is subjected to the additional axial forces induced by the brace.

3.3.2 Buildings retrofitted with cable tension braces

The same two frame buildings were retrofitted with steel cables as tension braces. Two types of steel cables were used for two types of tension braces in both buildings. The first tension cable consisted of 7-wire high-strength prestressing strands with 1860 MPa ultimate strength, placed diagonally in selected bays. The second type consisted of Grade 400 MPa mild-steel cables, also placed diagonally. Fig. 3-8 illustrates the models used for the analysis of buildings. The 6-storey building was retrofitted by bracing 4 bays in the critical short direction and the 2-storey building had two braces because of the lower seismic base shear observed in this building. The length of each strand was set to fit the diagonals between the lower and upper opposite corners, snug-tight, without any prestressing. The number and size of the strands were established through preliminary analyses using 2-dimensional plane frame analyses to save computational time, by subjecting them to the critical Cascadia CW3 record. The criterion used to establish the brace size was to reduce the maximum inter-storey drift to a level lower than the upper limit of 2.5% indicated in the 2015 NBC (NBCC (2015)). This resulted in the use of three different size cables for the 6-storey building, consisting of bundled or multiple strands to form steel areas that correspond to cables having a diameter of; i) 56 mm for the first floor, ii) 40 mm for the second and third floors, and iii) 10 mm for the upper floors. The use of multiple cables was validated in the experimental component of the same research program with anchors that permit the connection of multiple strands to the frame corners (Khosravi and Saatcioglu (2021)a). The 2-storey building was retrofitted with two cable sizes corresponding to a total nominal diameter of, i) 30 mm for the first floor and ii) 15mm for the second floor.

The buildings were modelled using the same material models used for the unretrofitted buildings, except for the addition of the cable material models. The steel model shown in Fig. 3-6 (b) was used for both types of steel, except that the stress-strain model used for the 1860 MPa 7-wire strands followed the stress-strain relationship obtained through coupon tests (Khosravi and Saatcioglu (2021)a) shown in Fig. 3-9. The models for the 6-storey and 2-storey buildings were constructed for response history analysis under the critical Cascadia CW3 record. The steel braces were defined as flexural elements with near zero flexural stiffness at both ends. They were modelled to be released under compression forces

so that they would resist only axial tension forces. The results for the first two stories are tabulated in Table 3-2 in terms of maximum storey drift, maximum axial tension in cable braces, and maximum axial force in Column C1. Hysteretic relationships obtained for the critical first-storey are plotted in Fig. 3-10. The results indicate that the total base shear resistance was increased when tension braces were used. The 6-storey buildings with 1860 MPa and 400 MPa cable braces developed maximum base shear resistances of 15,363 kN and 10,238 kN, respectively, in comparison with the total base shear capacity of 7,226 kN for the unretrofitted building, indicating significant increases in seismic force resistance due to retrofitting. Similar strength enhancements were observed in the 2-storey buildings with 1860 MPa and 400 MPa cable braces, with maximum base shear resistances of 5,599 kN and 3,935 kN, respectively, compared with the total base shear capacity of 3,029 kN for the unretrofitted building.

In addition to the strength increase, drift control was also achieved in the retrofitted buildings. The maximum first-storey drift in the 6-storey building was limited to 1.36% and 2.20% when 1860 MPa and 400 MPa steel braces were used, respectively, in comparison with over 5% drift attained in the companion unretrofitted building, which suffered from the failure of non-ductile frames. Similar observation could be made for the 2-storey low-rise building. In this case the maximum storey drift was reduced to 1.94% and 2.52% in when 1860 MPa and 400 MPa steel cable braces were used, respectively, as opposed to 3.69% lateral drift recorded in the unretrofitted building. It is clear from the foregoing discussion that both the strength improvement and drift control objectives were met in the retrofitted buildings. The buildings with 1860 MPa cables barely yielded during response, showing essentially elastic behaviour, while the buildings with 400 MPa cables developed yielding and energy dissipation but building failure was detected during deformation reversals beyond the first cycle at approximately 2% drift.

An important aspect of retrofitting buildings through lateral bracing is the effect of braces on nearby columns. Axial tension developed in the braces result in axial tension force components in nearby columns, which may offset gravity loads, causing potential uplift and further needs for column and foundation retrofits. Tables 3-1 and 3-2 lists maximum axial

forces experienced in Column C1, next to one of the braces. The computed column axial forces are shown in Table 3-2 for brace Arrangement 1, which is the X-brace arrangement that involves connections to the corners of the frame elements, as shown in Fig. 3-8, indicating substantial levels of axial tension in columns. Fig. 3-11 shows the variation of axial force in Column C1 during response for retrofitted and unretrfoitted buildings. While the negative effects of bracing can be minimized by spreading the braces over a larger number of bays, hence reducing forces per cable, additional options were considered and addressed in the following sections. These include the incorporation of SMA elements in cables and/or providing footings for the cables at the base, and supporting them from their own footings (Arrangement 2), thereby removing the burden on the adjacent columns.

3.3.3 Buildings retrofitted with steel-SMA assembly as cable tension braces

The cable brace system described in the preceding section resulted in strength enhancement and drift control with limited energy dissipation, though the mild steel cables yielded and dissipated energy, but developed early strength decay resulting in building failure during the 2% drift cycles. Both types of braces resulted in permanent deflections of the buildings upon yielding of the braces. Furthermore, there was a substantial increase in axial tension in the columns supporting the braces. To minimize these deficiencies of the retrofit system, shape memory alloy (NiTiCo) rods were included as part of the bracing system with flag-shaped hysteresis behaviour. Fig. 3-12 shows the analytical models used for the retrofitted buildings. Using SMA as a part of the steel cable braces has advantages, such as giving the brace the ability to undergo very large strains and returning back to zero deflection after unloading (self-centering capability). Also, it can reduce the base shear through inelastic behaviour.

Software “Seismostruct,” adopted as the dynamic analysis tool in the current research project, has a pre-defined model for cyclic behaviour of shape memory alloys, which makes modelling of the buildings relatively straight forward. The SMA model for cyclic behaviour is defined in the software to match the mechanical properties of the SMA rods tested by Mirzaeifar et al. (2011). The observed test results and the hysteretic model for SMA are shown in Fig. 3-13. This model was implemented in tension braces that consisted of a

combination of 1860 MPa strands and SMA where the SMA was modelled as a truss element with links at the ends that release resistance to compression and bending moment.

The key point in modelling the steel-SMA combination is to find an optimum length and size for the steel cable and the SMA. The high price of SMA rod is a consideration in selecting its size. The tension force in the steel cable is required to remain elastic during seismic response in order to force the SMA bar to yield and behave in the plastic mode to fulfil its functions. If the length of SMA is more than the optimum length, the strain in SMA becomes less than what is needed to fully utilize its strain capacity. If the length is too short, then the SMA may exceed its maximum strain capacity, which was established to be 8% in the current application. The safe strain for SMA in tension is considered to be 6%. Preliminary 2-D analysis was conducted to establish the lengths and sizes of brace components. Accordingly, the length of SMA rod was established to be 1.2 m and 1.0 m for 6-storey and 2-storey buildings with 70 mm and 30 mm diameters, respectively. The 1860 MPa steel cables had 56 mm and 30 mm diameters for the first floors of 6-storey and 2-storey buildings, respectively. The size of the other floor braces is the same as the snug-tight cable braces mentioned in the previous section. The other option for the steel cable component could be mild steel, to be used as mild-steel – SMA combination. However, care should be exercised to ensure that the SMA can develop its yield capacity prior to the yielding of the mild steel, as mild steel typically has lower yield strength than SMA.

The hysteretic responses of buildings retrofitted by cable braces having 1860 MPa steel-SMA combination, and their comparisons with those without the SMA rods are presented in Fig. 3-14. The incorporation of SMA rods in tension braces resulted in reduced base shear demands and increased energy dissipation while maintaining drift control. Furthermore, the presence of SMA promoted self-centering of the building after having experienced inelastic deformations. It further eliminated pinching of hysteresis loops associated with permanent plastic displacements of steel cables, which was more evidenced in yielding 400 MPa mild-steel cables. The same pinching is likely to appear in buildings with 1860 MPa cables without the SMA if the seismic force demands exceeded the yield capacity of the cables as it was observed during the frame tests (Molaei and Saatcioglu

(2014)).

Further assessment of braces with SMA is provided by inspecting the response of the steel and the SMA in retrofitted buildings. Fig. 3-15 shows the axial force-axial strain relationships for the 1860 MPA steel and the SMA rods. As can be seen, the steel strands remained elastic during response, while the SMA developed yielding and flag-shaped hysteretic relationship, dissipating significant energy and returning back to zero displacement upon unloading, promoting self-centering capabilities for the retrofitted buildings. For the 6-storey building, the maximum strain in steel strands was 0.0036, all remaining elastic. The maximum strain in the SMA rods was 5.6%, which lead to higher storey drifts and energy dissipation while maintaining the required drift control to protect non-ductile gravity frames in the building. When brace axial forces are compared, it can be observed that the use of SMA resulted in reduced brace forces with forces reducing from a maximum axial force of 2,839 kN to 1,943 kN. This is significant as the brace forces have direct implications on increased base shear and column axial forces. Although the use of SMA resulted in reduced base shear and axial force demands in braces and first-storey columns, further reduction of force demands may be necessary for buildings that have fewer high-capacity braces than larger number of smaller size braces sharing total seismic resistance. Therefore, further investigation of the tension brace system with SMA was investigated by considering a different arrangement of braces, as discussed in the next section.

3.3.4 Buildings retrofitted with steel-SMA assembly under different cable arrangements

The bracing arrangement used in previous applications involved connecting tension braces to column ends (Arrangement 1 shown in Fig. 3-8). This type of connection resulted in high shear forces and axial tension in the first-storey columns. Though the use of SMA rods reduced these force demands, further reduction may be necessary if the resulting forces exceed the column and column footing capacities. Therefore, a second set of analyses were conducted on the 6-storey building using the brace arrangement illustrated in Fig. 3-16 (Arrangement 2). This arrangement involved transferring the additional lateral force

resistance provided by the braces to the ground by anchoring braces to their own footings. A similar connection was provided for the second-storey braces. In this case the braces were extended to their foundation. By doing this, the length of the brace increased, developing smaller strains, tensile force and lateral stiffness. The same bracing mechanism may be considered for the third-story columns if tensile forces are experienced at this level of building. But usually, because of the relatively low seismic forces imposed at this level, it may not be necessary to apply the same strategy to the third-floor braces, or to the floors above.

The maximum response quantities obtained from Arrangements 1 and 2 are compared in Table 3-2. The results show the effectiveness of Arrangement 2, by eliminating tension forces in Columns C1 and C2, and by reducing the base shear at column C1 and the total base shear of the system.

3.4 Conclusions

The following conclusions can be drawn from the analytical investigation conducted on the use of tension cable systems involving the use of high-strength 7-wire strands and mild steel cables, as well as 7-wire strand-SMA combinations as seismic retrofit systems.

- Pre-1971 non-ductile reinforced concrete frame buildings located in regions of high seismicity in Canada lack seismic force and deformation capacities required by modern buildings codes based on the current seismicity, as well as the current design and detailing requirements. These buildings, when subjected to current code level seismic forces, suffer from excessive inelastic storey drifts and associated damage.
- Seismically deficient older buildings can be retrofitted by means of tension cable braces provided in selected bays in two diagonal directions. Tension cables may consist of 7-wire 1860 MPa prestressing strands, or 400 MPa mild steel cables.
- When 7-wire strand are used as tension braces, they provide effective drift control and increased lateral shear resistance. However, when attached to the corners of the frames, they may induce excessive axial tension and compression on the adjacent columns, potentially resulting in uplift and/or early crushing of column concrete. They

may dissipate limited energy but develop permanent deformations.

- When Grade 400 MPa mild steel is employed as tension braces, significant yielding and energy dissipation may occur in the cable, while drift control and increased lateral force resistance may be realized. However, because Grade 400 MPa steel may experience early yielding due to its relatively low yield strength, they must be designed with care so that the non-ductile frames in the system do not experience failures as the cables yield and develop higher drift demands.
- An alternative to steel cable braces is the use of the combination of steel cables with SMA bars. The flag-shaped hysteretic response of SMA results in significant dissipation of energy, self-centering capabilities that enable the recovery of plastic deformations, minimizing pinching of hysteresis loops. This system can also generate drift control and increase seismic force resistance.
- While the use of tension cable systems is easier to construct with less interference with the existing structure, it may cause significant axial tension and compression in the attached columns. This can be minimized by using smaller number of cables in larger number of bays, or by anchoring the cables to newly created footings at the ground level, transferring cable forces directly to their foundation, rather than imposing them on the nearby columns.

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3.6 Tables and Figures

Table 3-1 Summary of dynamic analysis results for un-retrofitted buildings

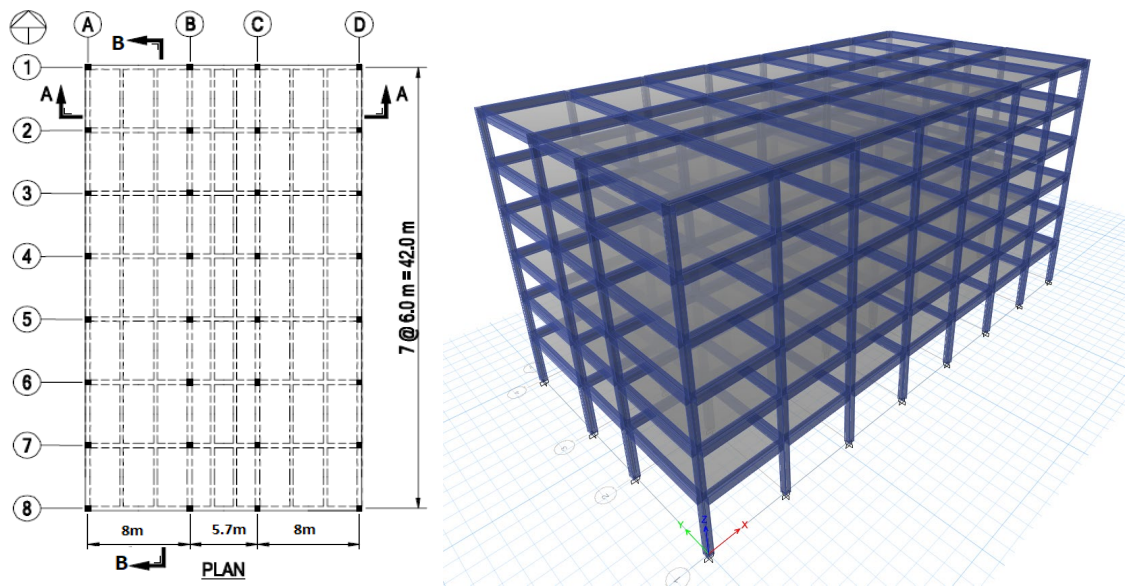
EQ Records	Time History Record	Mid-rise (6-Stories)			Low-rise (2-Stories)		
		Maximum Lateral Displacement (mm)	First Storey Lateral Drift (%)	Max Base-Shear (kN)	Maximum Lateral Displacement (mm)	First Storey Lateral Drift (%)	Max Base-Shear (kN)
Cascadia	CW1	142	3.15	7170	147	3.26	2932
	CW2	98	2.17	6734	77	1.72	2725
	CW3	220	>5	7226	166	3.69	3029
	CW4	83	1.84	7003	81	1.81	2769
Long Records	LW1	47	1.04	5906	59	1.3	2641
	LW2	72	1.59	6518	59	1.3	2645
	LW3	63	1.4	6391	59	1.3	2692
	LW4	68	1.5	6640	56	1.25	2644
Short Records	SW1	62	1.38	6757	43	0.95	2110
	SW2	27	0.6	4240	73	1.62	2801
	SW3	26	0.57	4031	60	1.34	2572
	SW4	41	0.9	5752	50	1.11	2264

Table 3-2 Maximum response quantities in retrofitted buildings

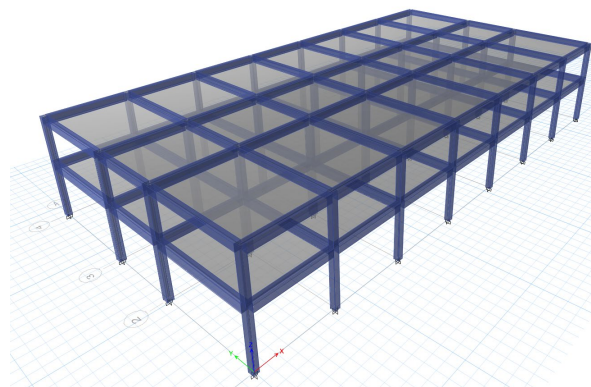
Building Type	Tension Cable Braces	Brace Arrangement	Max. Base Shear (kN)	Max. Column C1 Axial Tension Force (kN)*	Max. Cable Stress (MPa)		Max. Storey Drift (%)	
					1 st Floor	2 nd Floor	1 st Floor	2 nd Floor
6 Storey	1860 MPa Strands	Arrangement 1	15363	2839	1052	1001	1.36	1.59
6 Storey	400 MPa Mild Steel	Arrangement 1	10238	211	408	404	2.25	1.37
6 Storey	1860 MPa Strand With SMA	Arrangement 1	11810	1943	687	439	2.78	1.56
6 Storey	1860 MPa Strand With SMA	Arrangement 2	6499	-313	687	262	2.78	1.56
2 Storey	1860 MPa Strand	Arrangement 1	5599	646	1859	770	1.91	1.06
2 Storey	1860 MPa Strand With SMA	Arrangement 1	3729	-103	387	455	2.05	0.6

* Positive value for axial load indicates tension.

** For unretrofitted six storey and two storey buildings, the maximum axial tensile force are -1030 and -294 Respectively



(a) Plan view of Selected building (b) ETABS Model for the 6-storey building



(c) ETABS Model for the 2-storey building

Figure 3-1 Floor and elevation view of selected buildings

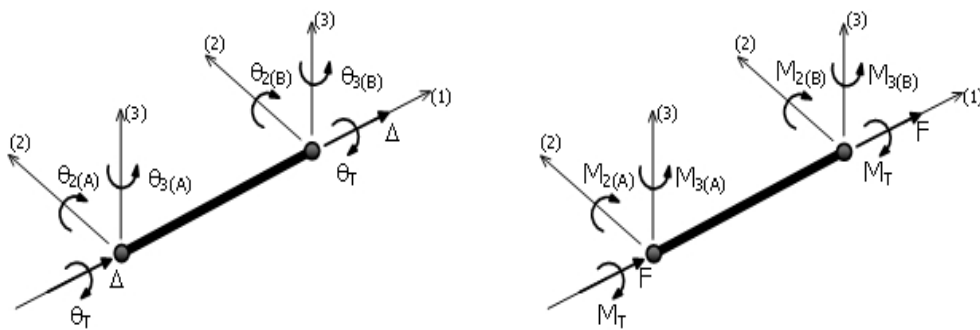
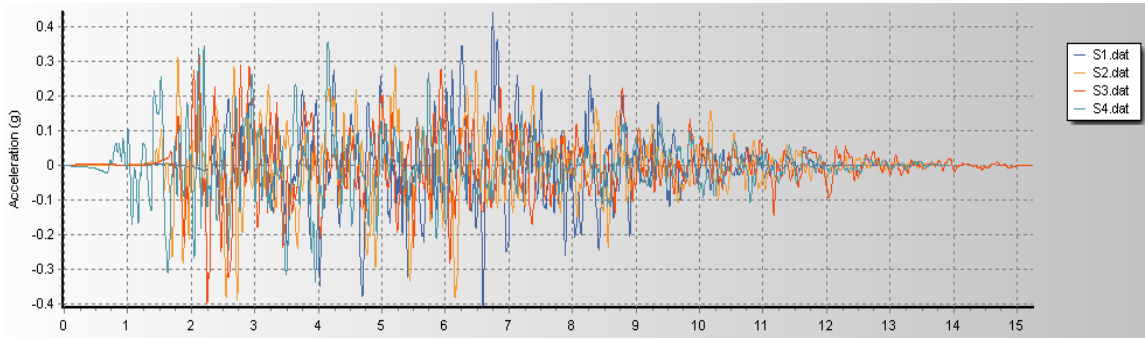
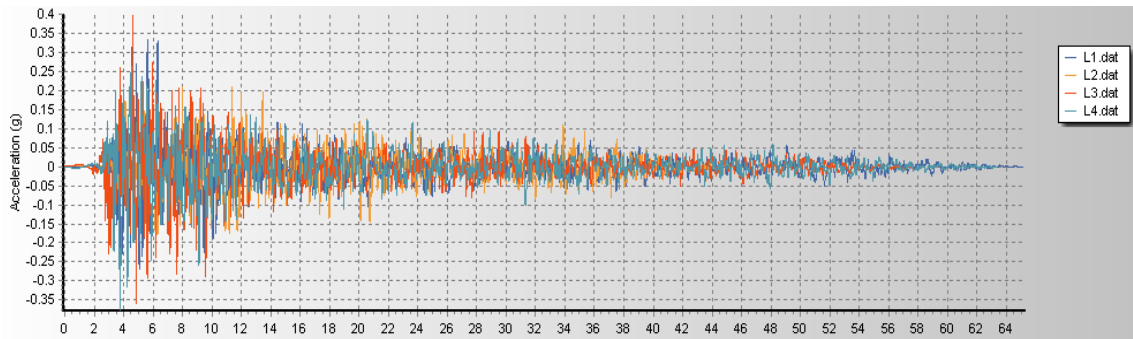


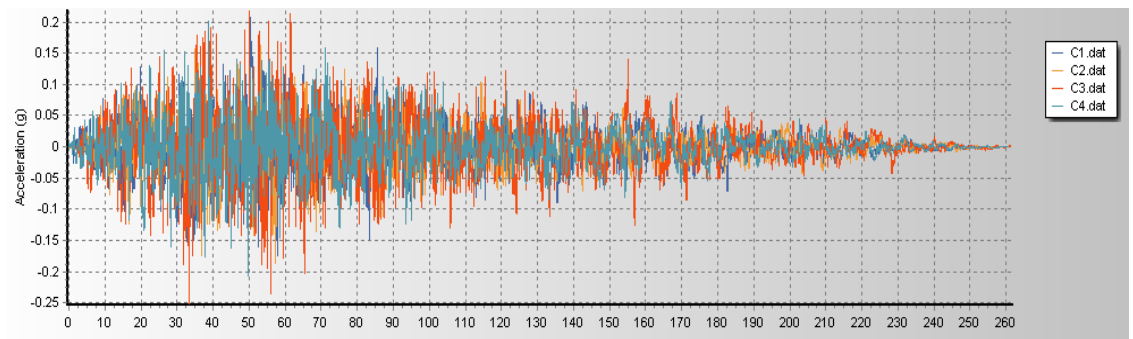
Figure 3-2 Degrees of freedom at both ends of an elements



(a) Short records

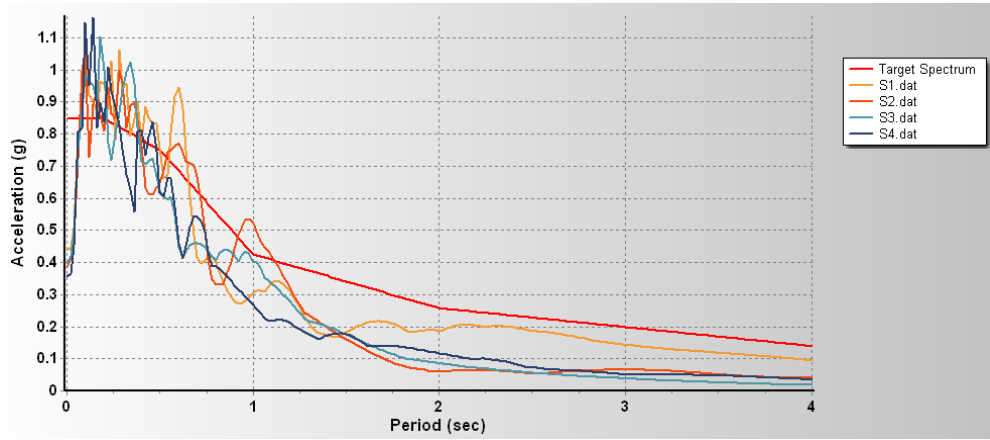


(b) Long records

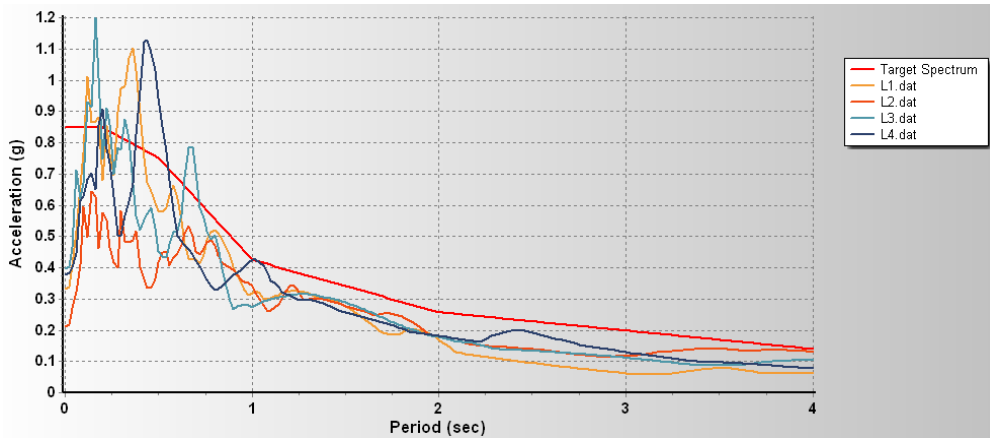


(c) Cascadia records

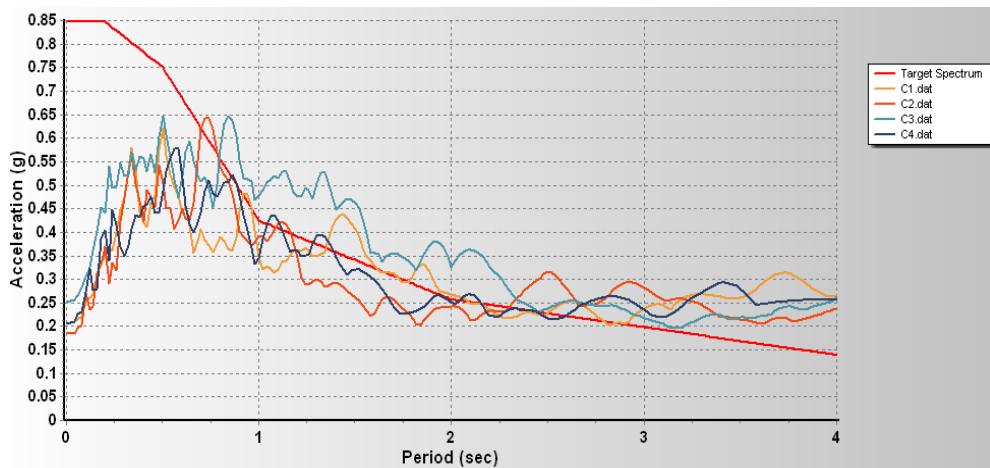
Figure 3-3 Artificial seismic time history records for Vancouver used in the analysis



(a) Short records



(b) Long records



(c) Cascadia records

Figure 3-4 Comparisons of response spectra with target design spectrum of NBC 2015 for Vancouver

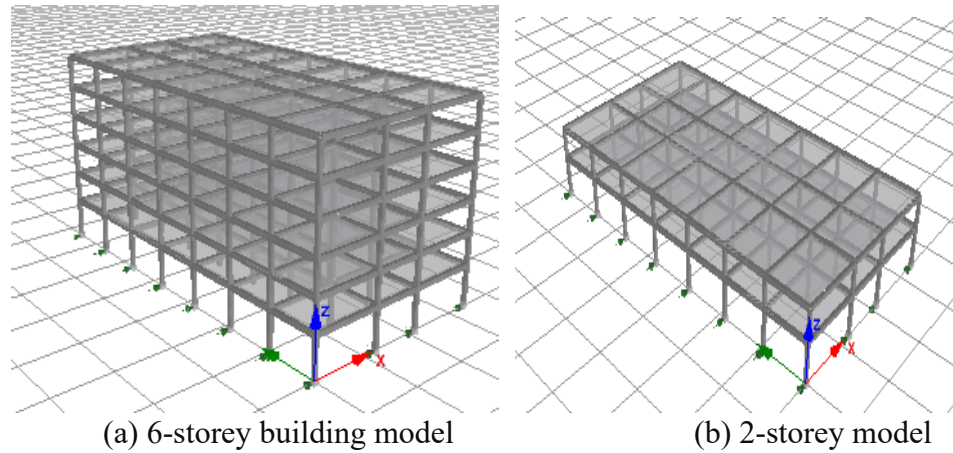


Figure 3-5 Analytical models of un-retrofitted buildings

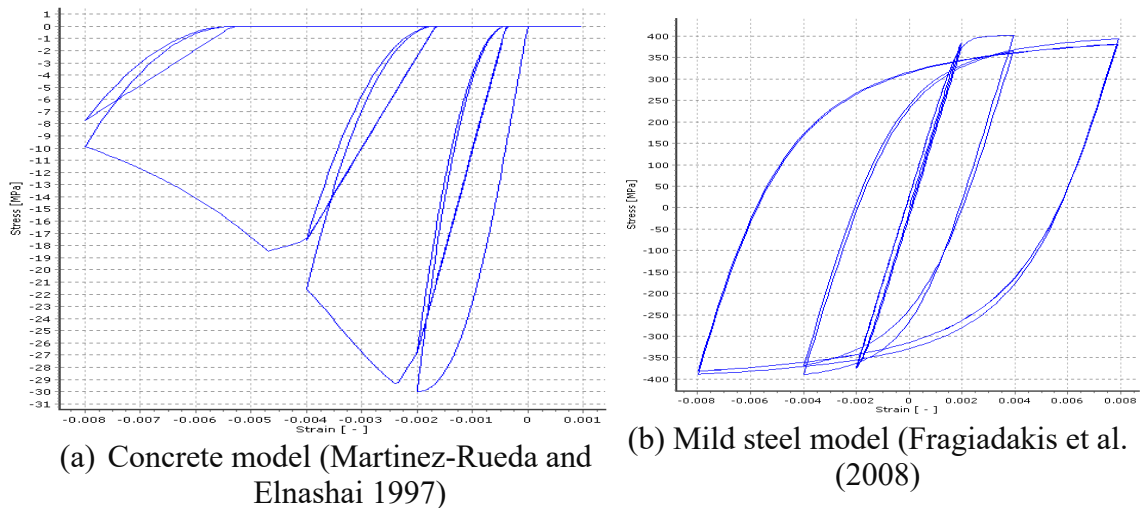
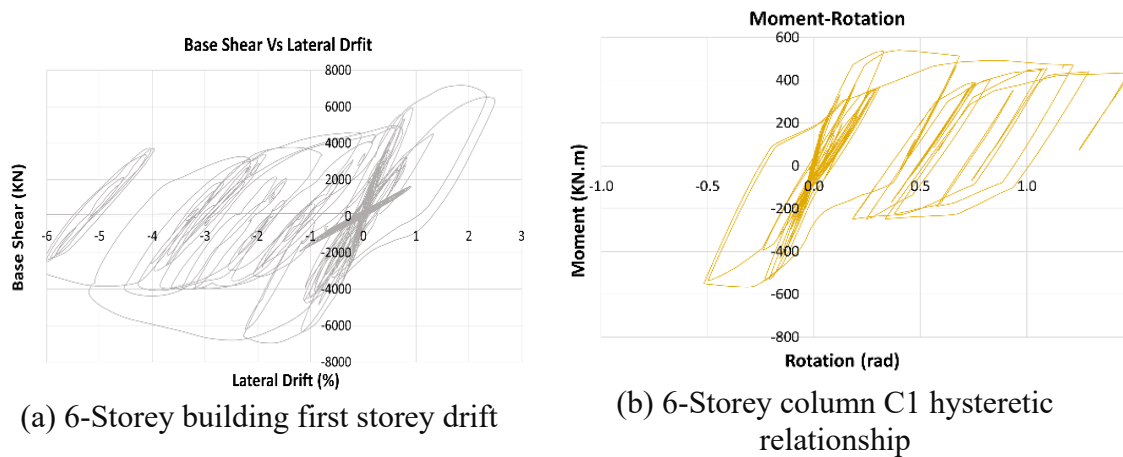
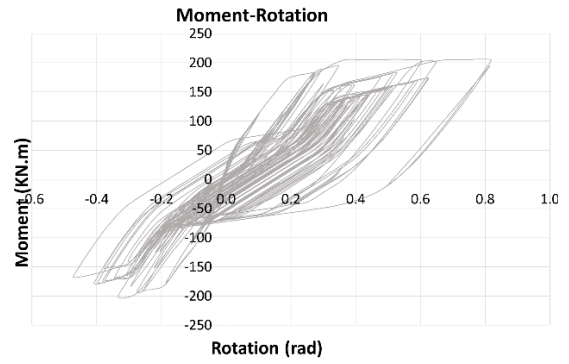
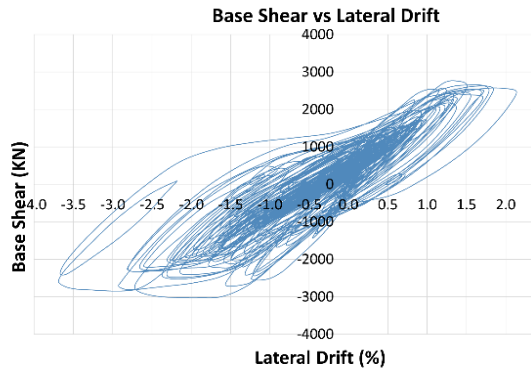


Figure 3-6 Material models used for frame elements

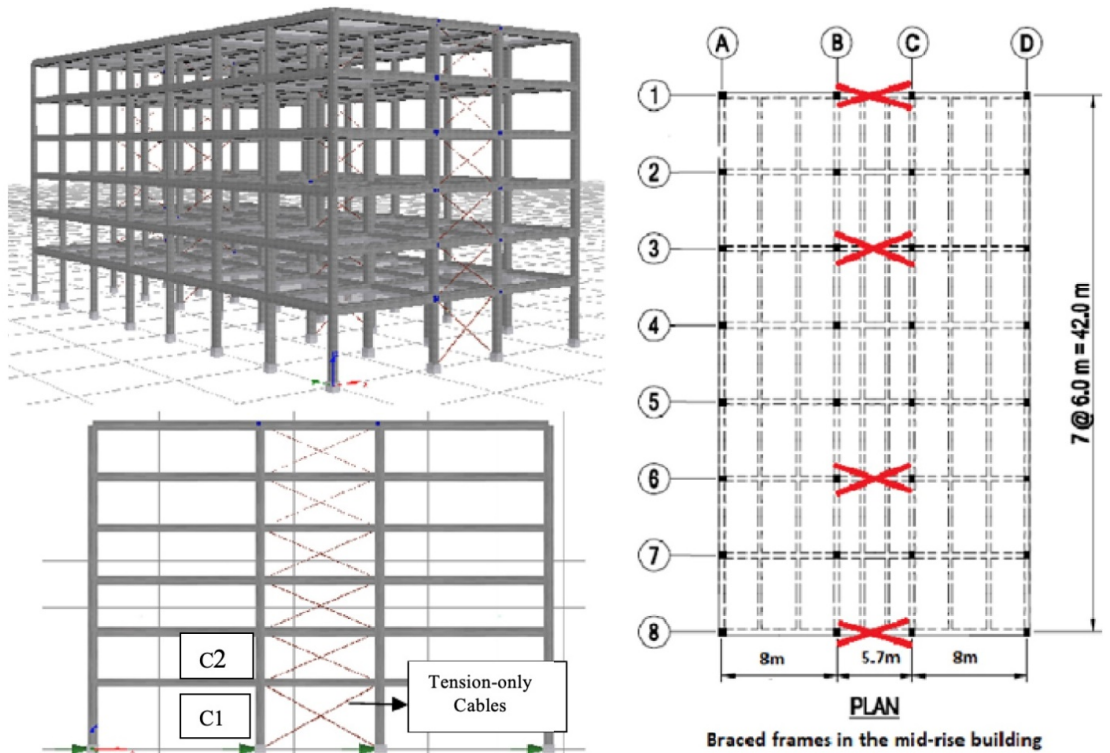




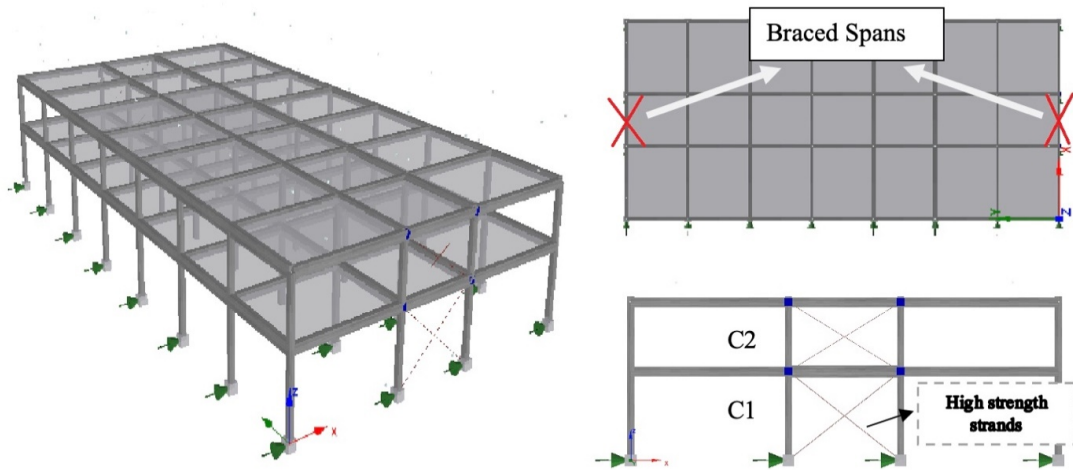
(c) 2-Storey building first storey drift

(d) 2-Storey column C1 hysteretic relationship

Figure 3-7 Hysteretic response of un-retrofitted buildings



(a) Retrofitted 6-storey building



(b) Retrofitted 2-storey building

Figure 3-8 Buildings retrofitted with steel cables

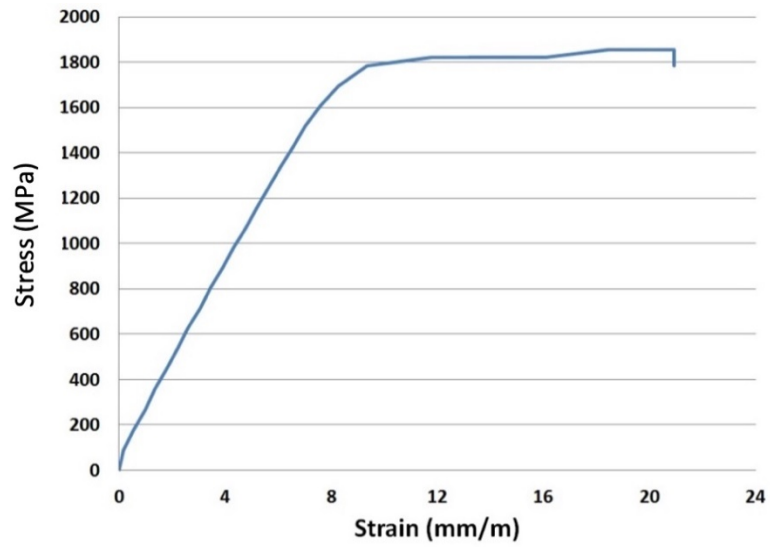
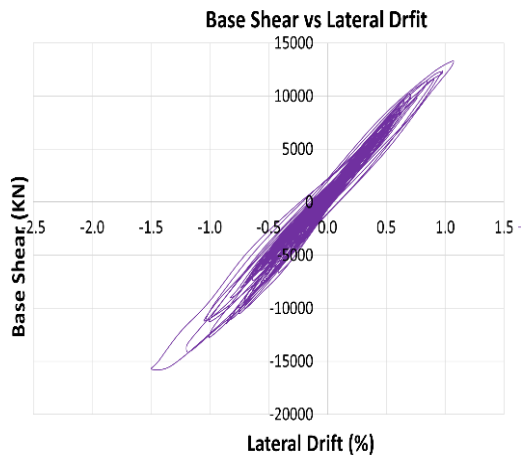
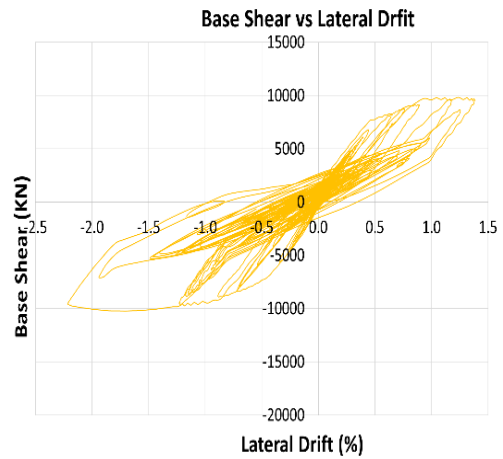


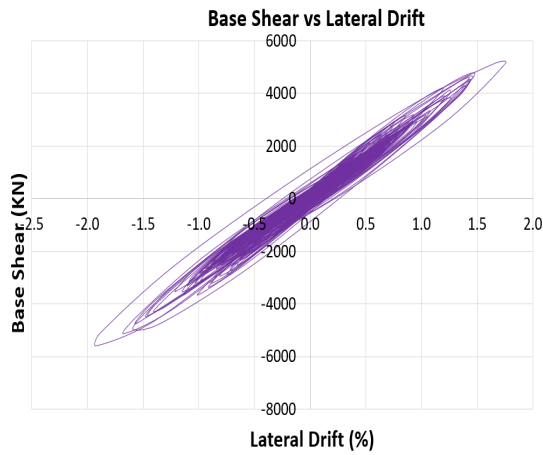
Figure 3-9 Stress-strain relationship of 7-wire strands



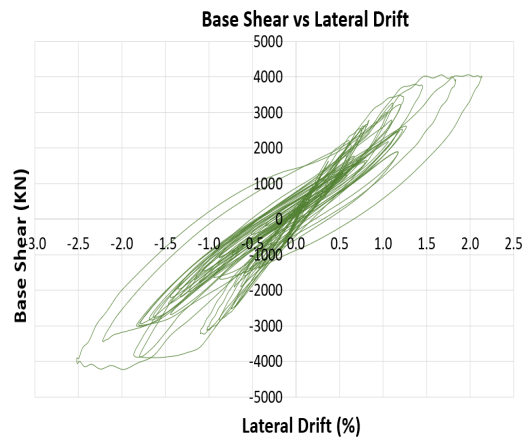
(a) 1860 MPa cable braces; 6-storey building



(b) 400 MPa cable braces; 6-storey building

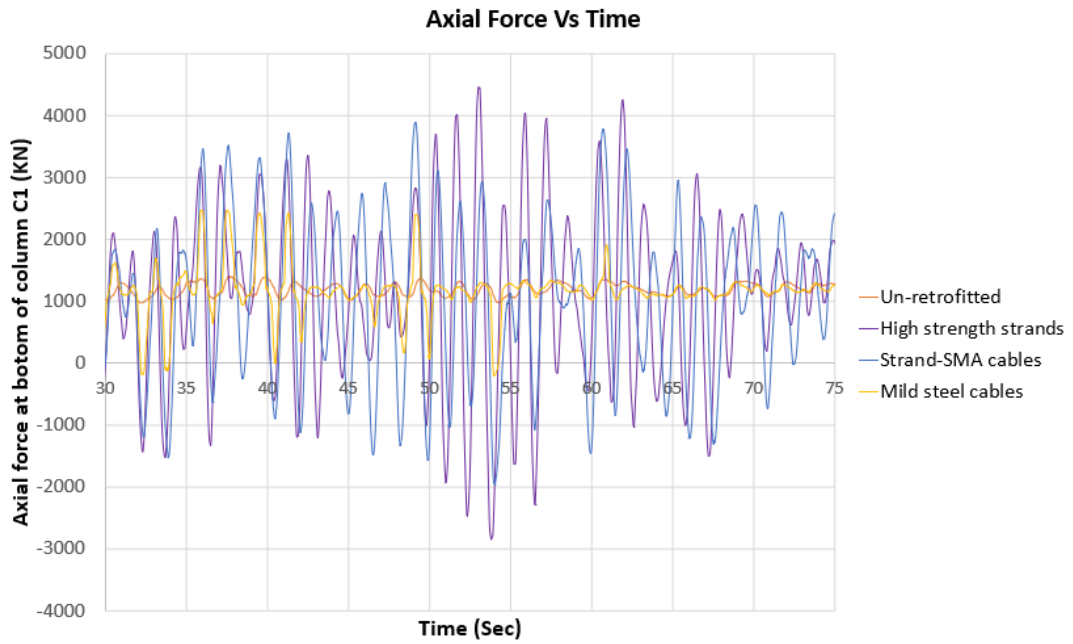


(c) 1860 MPa cable braces; 2-storey building

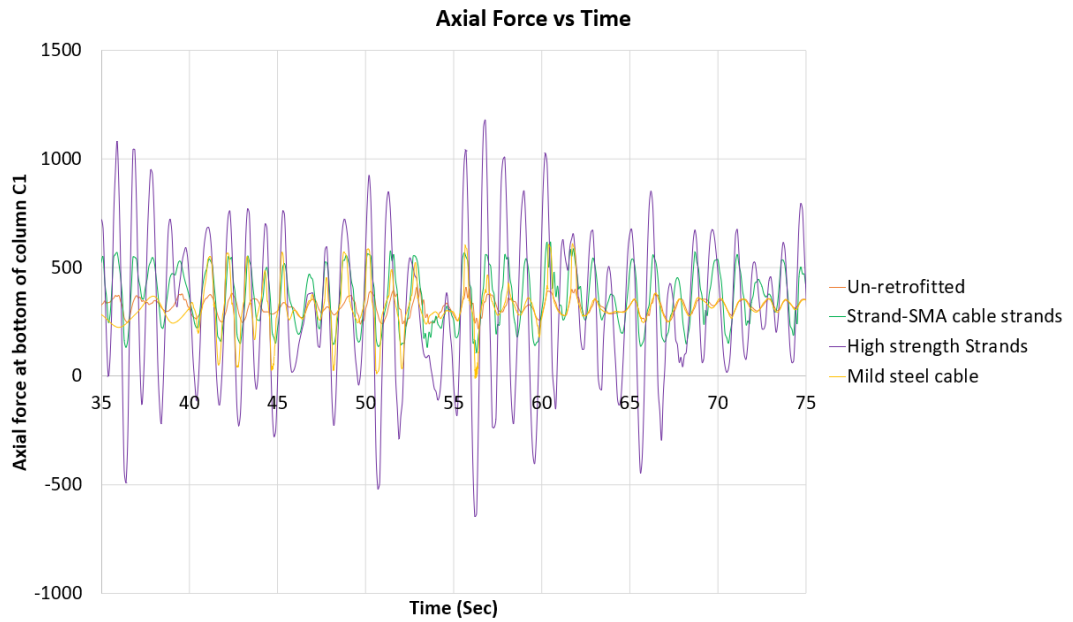


(d) 400 MPa cable braces; 2-storey building

Figure 3-10 Base-shear versus first-storey drift hysteretic relationships for steel cable retrofitted buildings



(a) 6-Storey building



(b) 2-Storey buildings

Figure 3-11 Time history of axial force variation in Column C1 at the base

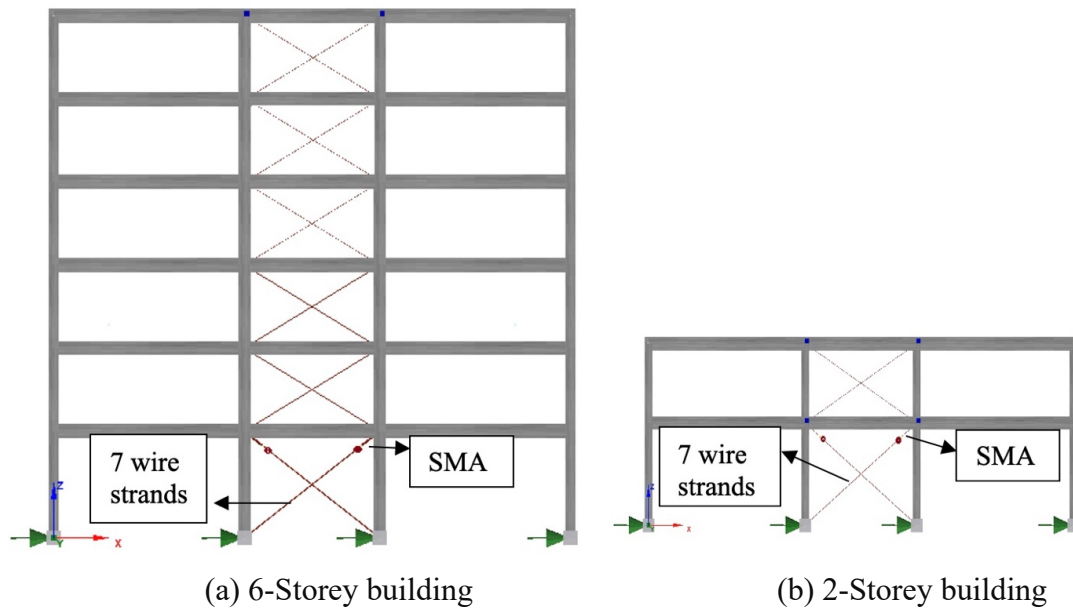


Figure 3-12 1860 MPa cables with SMA

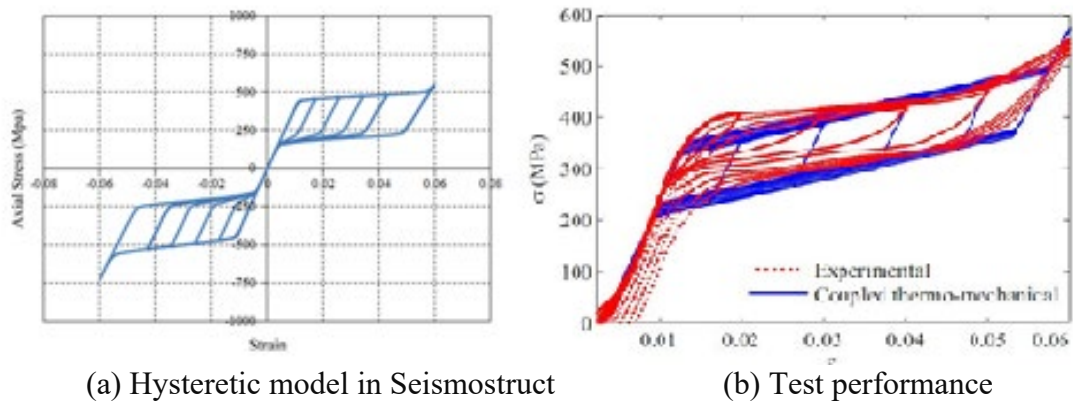
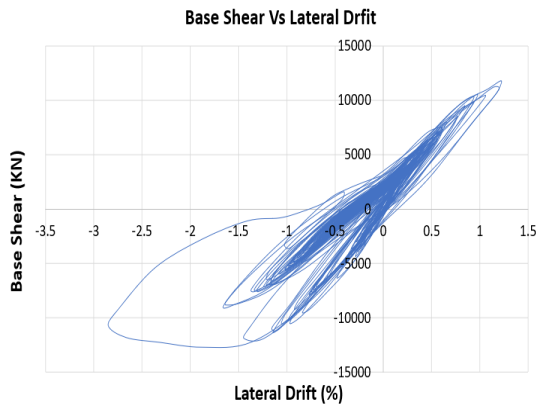
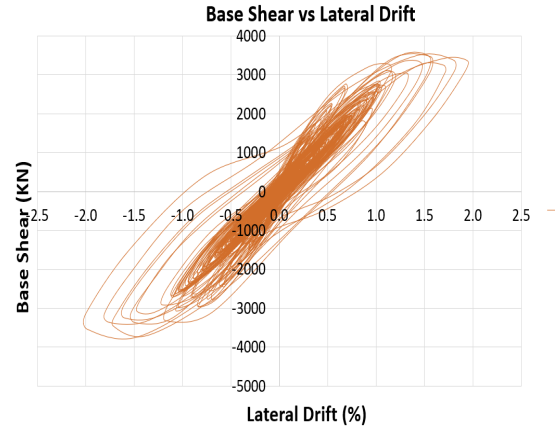


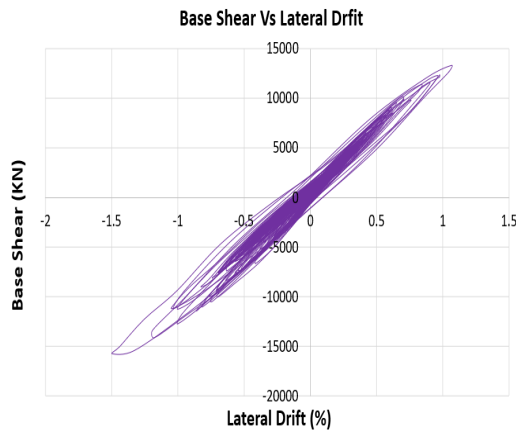
Figure 3-13 Hysteretic behaviour of the SMA



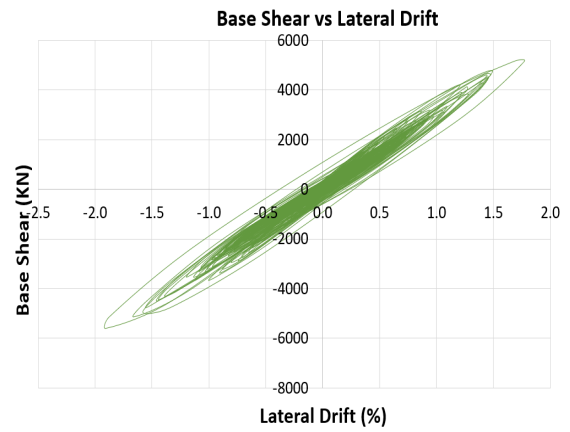
(a) 6-Storey building retrofitted with 1860 MPa-SMA combination



(b) 2-Storey building retrofitted with 1860 MPa-SMA combination

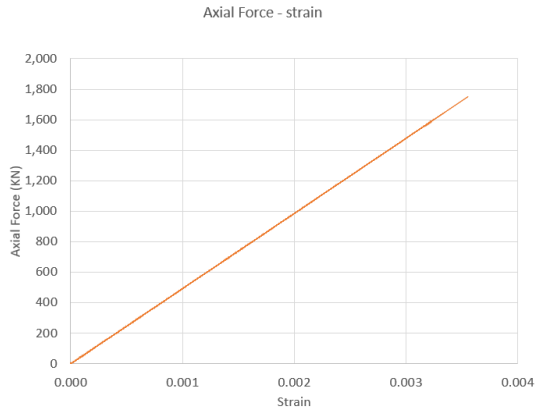


(c) Comparisons of 6-Storey buildings retrofitted by steel cables with and without SMA rods

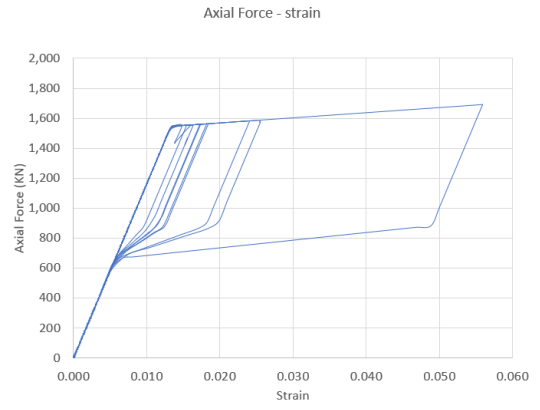


(d) Comparisons of 2-Storey buildings retrofitted by steel cables with and without SMA rods

Figure 3-14 Base-shear versus first-storey drift hysteretic relationships for buildings retrofitted by steel cables with and without SMA rods

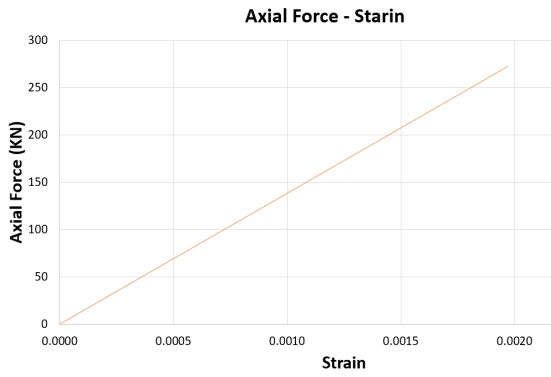


1860 MPa steel strands

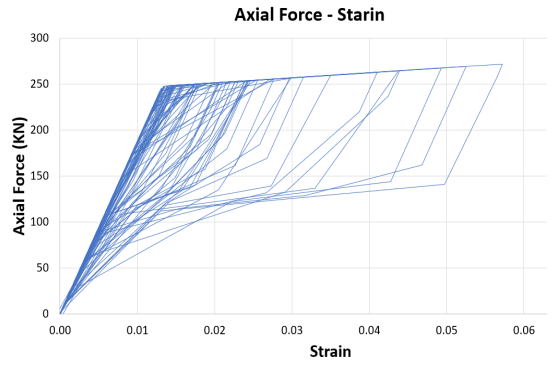


SMA

(a) 6 Storey building brace components



1860 MPa steel strands



SMA

(b) Storey building brace components

Figure 3-15 Axial force-axial strain hysteretic relationships for a first storey brace

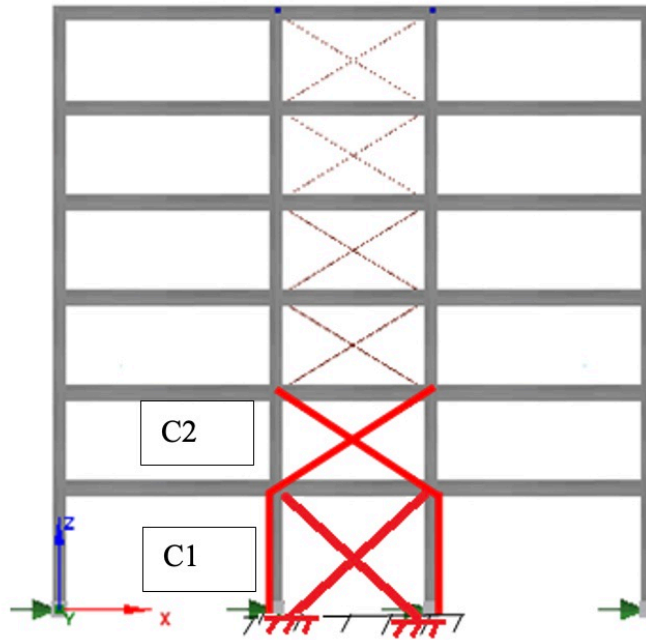


Figure 3-16 Brace Arrangement 2 for 6-storey building retrofitted by 1860 MPa cables with SMA

CHAPTER 4

4 Progressively Engaging Cables as Tension Braces for Seismic Retrofit of Non-Ductile Concrete Frames

Abstract:

Older reinforced concrete frame buildings, designed prior to the implementation of modern seismic codes are often seismically deficient in lateral force capacity and inelastic deformability. A common seismic risk mitigation strategy for such buildings involves the addition of bracing elements to improve lateral strength and stiffness. This results in reduced lateral drift, thereby controlling damage to non-ductile frames and non-structural elements as the existing frames continue fulfilling their gravity load carrying functions within their ductility capacities. Conventional lateral bracing systems include adding new reinforced concrete shear walls or steel braces of different types in selected bays. However, the increase in structural stiffness results in the shortening of the building period and associated potential increase in seismic force demands. Therefore, the bracing elements are designed to provide resistance to increased seismic force demands while also possessing inelastic deformability. Although this is a common practice to mitigate risk against design level earthquakes with long return periods, the structures in seismic regions are often subjected to more frequent lower intensity seismic excitations. A new system of lateral bracing was developed in the current paper, consisting of tension cable braces that controls the increase in seismic force demands. This is done by progressively engaging in lateral load resistance, thereby minimizing the increase in seismic force demands until the frame capacity is approached. Combined experimental and analytical research is presented based on a large-scale reinforced concrete frame, selected to represent pre-1970's North American seismic design practices. Frame tests were conducted to prove the concept, complemented by inelastic frame analysis.

Keyword: Concrete frames, bracing, cables, ductility, drift control, earthquake engineering, reinforced concrete, seismic design, seismic retrofit.

4.1 Introduction

Older reinforced concrete buildings designed and built prior to 1970s often lack sufficient strength and deformability against strong earthquakes. These buildings were essentially designed to resist gravity and wind loads without much emphasis placed on seismic design and detailing. Hence, they exhibit non-ductile or partially ductile response when subjected to strong earthquakes. A viable seismic risk mitigation strategy for such buildings is to implement seismic retrofit schemes to improve their seismic performance. Seismic retrofit techniques are implemented either at the element level, improving strength and ductility of seismically deficient elements, or at the system level, improving overall seismic force resistance of the entire structure while dissipating seismic energy and controlling lateral drift to minimize damage. The former approach is employed if few elements show lack of seismic design, and the latter is employed if the entire lateral force resisting system is deficient.

Conventional system level retrofit methodologies for reinforced concrete frame buildings involve lateral bracing of buildings by adding new bracing elements. Adding reinforced concrete shear walls in selected bays, well integrated with existing frame elements, is a common retrofit strategy. These walls serve as bracing elements, attracting a large proportion of seismic forces, controlling lateral drift because of their high rigidity. The addition of concrete structural walls has been researched extensively in the past and was shown to be an effective seismic retrofit methodology (e.g. Altin et al. (1992); Arslan et al. (2010); Canbay et al. (2003); Frosch et al. (1996). Similarly, structural steel braces can be added to existing frames to resist seismic forces and to control lateral drift. Many numerical and experimental research projects have been undertaken in the past, dating back to 1970s. Among the early researchers, Higashi and Kokusho (1975), Higashi et al. (1977), Higashi et al. (1980), Kawamata and Ohnuma (1981), and Sugano et al. (1980, 1981, 1983) showed proper repair and strengthening techniques for concrete structures. Other researchers who showed the effectiveness of steel braces in reinforced concrete frames include Badoux and Jirsa (1990), Bartera and Giacchetti (2004), Bush et al. (1991), Fukuyama and Sugano (2000); Ghobarah and Abou-Elfath (2001) and Ozelik and Binici (2008).

Another type of lateral bracing that gained popularity in recent years is the Buckling-Restrained Brace (BRB). BRB is a special type of concentric bracing, providing resistance in tension and compression with restraints provided against buckling in compression. This system was initially developed by Watanabe et al. (1988) and subsequently researched by others (Tremblay et al. 1999, 2006, 2011; Uang and Nakashima (2004); López et al. (2004), and Al-Sadoon et al. (2020)).

Researchers in the past also investigated compression only and tension only braces. Compression braces were investigated by Caron (2010) by testing a single storey-single bay frame. The objective was to avoid tension connections of X braces to older concrete joints. The use of tension only braces involved the use of diagonal steel cables or strands placed symmetrically in two directions so that they develop tension resistance under load reversals. Pincheira José and Jirsa James (1995) investigated the effectiveness of tension braces in buildings. This technique was researched and used in a limited manner after the 1985 Mexico City Earthquake. It has since been further investigated at the University of Ottawa through experimental and analytical research by Shalouf and Saatcioglu (2006), Carrière (2008) and Molaei and Saatcioglu (2014). The use of tension braces consisting of 1860 MPa strands, with and without prestressing, were shown to be effective in increasing strength and stiffness of concrete frame buildings.

One of the consequences of stiffening buildings with lateral bracing is to change the dynamic properties of buildings, resulting in the shortening of their periods. Buildings in the short period range often attract higher seismic forces. The increase in seismic force demands is offset by the increase in lateral load capacity through the added bracing elements, which are preferably designed with inelastic deformation capacity to contribute towards the energy dissipation of the retrofitted building. Another effect of lateral bracing is to increase the axial force in the attached columns. Axial tension in columns can be high enough to offset gravity loads, resulting in net tension. This requires further strengthening of the columns and their foundations, which may suffer from uplift, requiring foundation retrofits. Compression on the columns reduce column deformability, while promoting early concrete crushing. Controlling the increase in column axial load through the use of shape

memory alloys (SMA) and providing separate footings for the tension cables at the ground level has been addressed by the authors in an earlier phase of the current investigation Khosravi and Saatcioglu (2021)b. The objective of the current research project is to develop a new technique for the use of tension cable bracing systems to minimize seismic force demands and associated increase in column axial forces. This is done by introducing the concept of progressively engaging cables (PEC). Accordingly, multiple 1860 MPa high-strength prestressing strands are placed loosely as tension elements such that each strand engages in resisting lateral forces progressively when needed. The strands initially remain unstressed and do not participate in seismic response until the lateral drift becomes sufficiently high and the frame requires further lateral resistance. At this stage of loading the frame responds based on their original stiffness and period, without increasing seismic force demands. This stage of loading corresponds to more frequent low to medium intensity seismic excitations. If the seismic intensity becomes higher, the cables start engaging in lateral force resistance. The first of multiple strands is designed to engage at a prescribed drift level, depending on the available ductility of the building. As the first strand yields and dissipates energy, the second and possibly the third strands start engaging in lateral load resistance at different drift levels, while some strands possibly remain elastic, recovering some of the inelastic deformations, promoting self centering capabilities. The strands are designed to limit lateral drift so that the frames continue fulfilling their gravity load functions, even if they experience some controlled level of strength decay as the tension braces gradually take over lateral load resistance and provide increasingly higher bracing action to resist higher proportion of seismic forces.

A similar concept of delayed participation of tension braces were developed by Hou and Tagawa (2009), and Mousavi and Zahrai (2017) for steel frames. The former group of researchers used a cylindrical steel element in a central location of the X arrangement of tension cables, which allowed an initial slack in the system, delaying the development of tension forces in cables as illustrated in Fig. 4-1(a). The latter group used additional steel extenders tied to the cables at frame corners to leave slackness in the cables, equipped with turnbuckles for the adjustment of the cable slackness as illustrated in Fig. 4-1(b). Both groups of researchers applied the concept to scaled structural steel test frames and

demonstrated their effectiveness.

The concept of PEC for reinforced concrete frames was developed and illustrated in this paper through combined analytical and experimental research. A single bay-single storey frame, representing pre-1970s North American design practice, was selected for the investigation. Tests of the frame with different arrangements of PEC were conducted under simulated seismic loading. Non-linear analyses of the frames were conducted to illustrate the use of different PEC arrangements.

4.2 Selection of a reference frame

A 6-storey reinforced concrete frame building, designed and used by Al-Sadoon (2016) in his PhD research at the University of Ottawa as part of the same overall research program on seismic retrofit of concrete frame building, has been adopted in the current investigation. The building was designed using the 1965 edition of the National Building Code of Canada ((NBCC) (NRCC (1965))) as representative of seismically deficient buildings in Canada. Design and detailing of the frame elements followed the requirements of ACI 318-63, which was referenced in the 1965 NBC (NRCC (1965)) for designing reinforced concrete structures. A test frame was designed as a 2/3rd scaled model of one of the middle bays in the critical short direction of the prototype building (Al-Sadoon (2016)). The design was based on scaled load combinations, including dead load, live load, wind load and seismic load for Vancouver, Canada. The seismic base shear was calculated using the importance factor of 1.0 as a building with normal importance, and non-ductile construction type with factor $C = 1.25$. The foundation factor F was assumed to be 1.0, implying that the building was built on a soil condition that was not highly compressible. A plane frame model was built in computer software SAP 2000 (CSI 2010) and the design was carried out using the software. The resulting geometry and reinforcement detailing of the frame are illustrated in Fig. 4-2. The columns were supported by an I shaped foundation to ensure full fixity to the laboratory strong floor during testing. It is noteworthy that the beam-column joints did not have transverse reinforcement since it was not a requirement in the NBC 1965 (NRCC (1965)). The transverse reinforcement for columns consisted of 10M (11.3 mm diameter) ties spaces at 200 mm c/c as per the design code.

4.3 Dynamic inelastic analysis of test frames

A series of non-linear dynamic analyses of the test frame was conducted to establish the parameters of the experimental program. A number of available software packages were considered first for the analyses. Among the software considered, SeismoStruct (2016) was selected for the analyses, as it was used in the previous phase of the same research program due to the suitability of its hysteresis models Khosravi and Saatcioglu (2021)b.

4.3.1 Software and frame data specified for dynamic inelastic analysis

SeismoStruct (2016) is a fibre-based structural analysis program with emphasis on nonlinear behaviour. It is a powerful structural finite element software that can simulate structural behaviour under static and dynamic loads while considering both geometric and material nonlinearities. In addition to dynamic and static time history analysis options, this software is capable of running conventional and adaptive pushover analysis, response spectrum analysis, incremental dynamic analysis, eigenvalue analysis and variable static loading.

The software has element models suitable for beams and columns with geometric and material non-linearities. The flexural behaviour of structural sections is obtained through the integration of stresses and strains in individual fibers with nonlinear uniaxial material models assigned to each fiber. For the frame analyses, the concrete model was defined as a nonlinear element stress-strain relationship as defined by Madas (1993), also incorporating the effect of confinement as proposed by Mander et al. (1988). The cyclic behaviour of concrete was modelled as proposed by Martínez-Rueda and Elnashai (1997). Fig. 4-3(a) shows the concrete model. The confinement effect was calculated based on the size and spacing of stirrups with constant confining pressure throughout the entire stress-strain range. Strain corresponding to the peak stress of unconfined concrete (f'_c) was considered to be equal to 0.002 mm/mm. The tensile strength of concrete was specified for tension generated under different stress conditions such as flexural ($0.6\sqrt{f'_c}$) or direct tension ($0.33\sqrt{f'_c}$). When maximum tension resistance was reached, the concrete was assumed to lose its tensile resistance without any tension softening. The modulus of elasticity of concrete was defined as the initial elastic stiffness of the material and set to 25,743 MPa for the 30MPa concrete

used for the modeling of the test frame. The mass density of concrete was considered to be 2400 kg/m^3 . The reinforcing steel followed a uniaxial stress-strain model for mild steel. It was based on the stress-strain relationship proposed by Menegotto and Pinto (1973), followed by the isotropic hardening rules proposed by Filippou et al. (1983). The cyclic seismic loading rules recommended by Fragiadakis et al. (2008) was incorporated as illustrated in Fig. 4-3(b). The parameters that defined the mild steel model included the initial elastic modulus of elasticity, yield strength, the strain hardening parameter, (E_{sp}) and the initial elastic stiffness (E_s). The yield strengths for 20M (19.6 mm diameter), 15M (16 mm diameter) and 10M (11.3 mm diameter) reinforcing bars were established through coupon tests and were specified as 430 MPa, 450 MPa and 481 MPa, respectively. The Baushinger effect (R) was introduced through the transition curve initial shape parameter which controls the shape of the transition curve between the initial and post-yield stiffnesses necessary to simulate the Baushinger effect and the pinching of hysteresis loops. In addition, the fracture strain in tension and buckling strain in compression needed to be specified. The mass density of steel was taken to be $7,800 \text{ kg/m}^3$. The beam and column cross sections were meshed using 600 mesh per member. Fig. 4-4 illustrates a typical discretization of a column and a beam element.

The member response was obtained by fully accounting for the spread of inelasticity along the member length by integrating individual sectional response. This type of modelling is considered to be an accurate representation of flexural behaviour since it can consider inelastic behaviour along the entire length of a structural member. In order to have the maximum accuracy of the analyses it is recommended to have at least 200 fibers in each cross section. In addition to the section fibers, up to 7 integration sections may be needed to accurately model the hardening response for members. The Sismostruct (Seismosoft (2016)) Manual recommends 4 to 5 integration sections to ensure that the element reaches its softening response range. The software also considers plastic-hinging within a fixed length of the element (Scott and Fenves (2006)). The plastic hinge length should be defined by the user. Frame deformations are calculated considering inelastic behaviour of elements. P-Delta effects are incorporated into the analysis by calculating large displacements through the total chord rotational formulation developed and implemented by Correia and Virtuoso

(2006). The chord rotation formulation is based on an exact description of the kinematic transformations associated with large displacements and three-dimensional rotations of the beam-column member. This leads to the correct definition of element deformations and forces, as well as to the effects of geometrical non-linearities on the stiffness matrix. The implementation of this formulation considers, small deformations relative to the element's chord, notwithstanding the presence of large nodal displacements and rotations. In the local coordinate system of the beam-column element, six basic displacement degrees-of-freedom and corresponding element internal forces are defined, as shown in Fig. 4-5.

The tension braces were modelled as truss elements without any resistance in compression using a bi-linear stress-strain relationship. For the 1860 MPa 7-wire strands, the stress-strain relationship obtained by coupon tests was used. Accordingly, the yield point was defined as 1650 MPa at 0.9% of strain and the stress at ultimate strength was set at 1860 MPa corresponding to a strain of 2.1%.

The internal forces developed by inelastic frame elements are computed by the software. These values are correlated with axial forces, shear forces and moments (caused by flexural or torsional) at the end of each node. By finding both forces and deformations (rotations) at the ends of each element, the hysteretic behaviour is obtained in the form of force-displacement relationship.

4.3.2 Analysis results

Inelastic analysis of the test frame was conducted following the same loading protocol intended to be used during the experiments. The loading protocol, shown in Fig. 4-6, consisted of incrementally increasing lateral drift reversals with three cycles applied at each deformation level. The first analysis was conducted on the unretrofitted frame using the model shown in Fig. 4-7(a). The results of this analysis provide reference values for comparison with retrofitted frames. The subsequent analyses consisted of retrofitted frames. The analytical model for retrofitted frames is shown in Fig. 4-7(b). The tension braces were attached to the foundation to simulate the support condition of the test frames. The frames were first loaded with vertical point loads to simulate gravity loads. Two-point loads were

applied on two columns, 800 kN each, representing scaled gravity column loads applied on the prototype building. Two additional point loads were applied on the girder, each at 0.85 m from the face of each column with a magnitude of 62 kN representing scaled dead and live loads acting over the tributary area for the transverse beams that transfer the loads to the girder, simulating the experimental loading condition. The analyses were conducted under imposed lateral displacement conditions, following the loading protocol illustrated in Fig. 4-6. The lateral displacements were applied horizontally in the X direction.

The first retrofitted frame had a single snug-tight strand in each diagonal direction, which was design to engage in lateral load resistance as soon as the frame was loaded laterally. The length of the strands was set to fit the length from the top corners to the foundation with an appropriate angle with the horizontal to match the anchor locations in the foundation. The strand size was established to result in approximately a factor of 2.0 increase in seismic resistance. The strands in the analytical model were attached to the joints through gap link elements. Gap link was defined in a way that the strands worked only in tension, without resisting compression.

The hysteretic relationships obtained from the analyses of unretrofitted and retrofitted frames are shown in Fig. 4-8. The results indicate about a factor of 2.0 increase in seismic resistance with the use of snug-tight tension braces. The maximum lateral load capacity of the frame increased from 201 kN at 1.85% drift ratio to 397 kN at 2% lateral drift. The crushing of column core concrete in the retrofitted frame initiated at 2.3% drift ratio, followed by a rapid strength deterioration and collapse of the building at 3% lateral drift. Concrete crushing, which triggered collapse, was attributed to increased column axial compression due to the presence of diagonal braces. It was also observed that the tension cables yielded during response as depicted in Fig. 4-9, dissipating seismic energy but also developing permanent elongation, which led to the pinching of hysteresis curves.

The use of high-strength strands as tension-only braces resulted in a significant increase in seismic force resistance. However, the deformability of the frame did not improve. Furthermore, the increase in stiffness of the frame during seismic excitations is expected to shorten the period of the overall system, potentially attracting higher seismic forces in the

short period range as indicated by seismic response spectra. A seismically deficient frame may not need seismic retrofitting during moderate to low magnitude earthquakes in high seismic zones. Increasing the stiffness through bracing for such events may be more harmful than helpful because of the associated increase in seismic force demands. Furthermore, even under strong earthquakes, the interference of the braces in structural response can be delayed until the frame has developed significant softening upon cracking and initial yielding, reaching a critical drift level beyond which it requires lateral bracing. Therefore, progressively engaging cable (PEC) system as tension-only braces was explored as a new bracing system for reinforced concrete frames. This requires the use of loose strands that will engage in response at different drift levels. The strands could be a single strand in each direction, or multiple strands where each strand engages in response at different level of lateral drift. The mechanism of the method was explored through inelastic analyses of the test frame prior to conducting tests as proof of concept. This was done by modelling the strands with a gap link defining the initial slack. The frames were subjected to the same loading protocol used in the earlier analyses up to 4% lateral drift as illustrated in Fig. 4-6. The initial set of analyses involved the use of 15 mm diameter single loose strands engaging at different drift levels. Three different scenarios were considered with three different target drift ratios to activate the braces at; i) 1% drift with a cable gap of 21.5 mm, ii) 1.5% drift with a cable gap of 32.4 mm, and iii) 2% drift with a cable gap of 43.3 mm. Fig. 4-10 shows the analysis results in terms of hysteretic responses. The results indicate that all the frames with loose strands developed improved strength. Those that engaged in lateral force resistance at higher drifts showed higher deformabilities. The peak resistance of 367 kN was attained at 2.7% drift when the strand was activated at 1% drift (as opposed to 201 kN at 1.85% drift for the unretrofitted frame). The increase in strength was somewhat less when the activation of the strands was delayed, but the increase in deformability was higher (343 kN at 3.2% drift and 310 kN at 4% drift for cables that were activated at 1.5% and 2.0% drifts, respectively). The rate of strength decay beyond the peak was reduced as the activation of the strands were delayed. The hysteresis loops reflect the increase in stiffness at drift levels at which the lateral resistance of the strands was activated. It is also observed that the decay in lateral load resistance of the concrete frame beyond its capacity is offset

by the participation of the braces, thereby maintain a high level of resistance up to 4% lateral drift. This is illustrated in Fig. 4-11.

The next phase of analyses included multiple progressively engaging loose strands, with different percentages of activation drifts assigned. The same test frame was retrofitted with three 15 mm diameter high-strength 7-wire strands in each direction. The same modelling techniques as the previous models were employed. The first application was designed to have one strand engage in load resistance at 1% drift and the other two at 1.5%. The second and third applications had one of the three strands engage in 1.5% and 2.0% drift ratios, while the other two engaging at 2% and 3% drifts, respectively. This technique led to initially low seismic force demands and reduced brace contributions, followed by increased resistance, while stiffening the frame and potentially increasing seismic force demands at a later stage of load when the frame has already experienced some level of softening. The hysteretic relationships obtained from all three analyses are shown in Fig. 4-12. The results are consistent with the observations made for the use of single loose strands earlier, except for the increased level of force resistance because of the use of multiple strands. The deformability of retrofitted frames improved as the engagement of the cables was delayed at the expense of developing lower resistance. The number of strands and their activation drift levels need to be decided in practical applications based on the nature of the existing non-ductile frames and the required enhancements in strength and deformability. In the test frame analyzed in this section, the strength decay in the unretrofitted frame began at about 2% lateral drift. Therefore, it may be feasible to delay the engagement of the first strand to 1.5% or 2% drift. This decision will depend on the level of strength and ductility enhancement expected from the frame to be retrofitted. It was observed in the analyses that the first cable that was engaged in lateral load resistance always yielded, while the others remained elastic, though some did approach yielding. The axial force-axial deformation relationships for the strands are shown in Fig. 4-13 for all of the multi-strand braces.

The objective of the analyses of the test frame prior to experimental research was to decide on the parameters of the test program. Based on the analyses presented, it was decided to have two frame tests, one with a single 15 mm diameter 7-wire strand engaging

at 1% lateral drift and the other with three strands of the same size, first the first strand engaging at 1.5% drift and the other two engaging at 2% lateral drift. The details of the experimental research are presented in the following section.

4.4 Experimental Research

The 2/3 scale single bay-single span test frame, described earlier with geometric details shown in Fig. 4-2, was used in the experimental phase of research. The frame was cast using ready-mixed concrete with 10 mm aggregate size. Standard cylinders were cast and tested to establish the average concrete strength as 31 MPa. Standard tension coupon tests were conducted to establish the mechanical properties of deformed reinforcing bars used in the frame specimen. The average yield strengths for 10M, 15M, and 20M reinforcing bars were established to be 481 MPa, 450 MPa, and 430 MPa, respectively. Fig. 4-14(a) provides typical stress-strain relationships for each type of reinforcing bar used. Coupon tests were also conducted on the seven-wire high-strength prestressing strands used to apply the gravity loads, as well as forming the PEC elements as tension only X bracings. The strands were #15 7-wire prestressing steel with a nominal diameter of 15.24 mm and a nominal cross-sectional area of 142 mm². The ultimate strength of the strands was specified to be 1860 MPa by the manufacturer. The yield strength, as established by 0.2% offset method was 1650 MPa. The tensile rupturing strength was observed to be 1857 MPa. Fig. 4-14(b) shows the average stress-strain relationship of 7-wire strands.

The details of the experimental research, including specimen preparation, instrumentation, test setup and test procedure are presented in the following sections.

4.4.1 Preparation of the test frame for testing

Fig. 4-15(a) shows the frame shortly after it was cast. The frame was first loaded by gravity loads by means of vertical prestressing strands. Six high strength steel strands were used to apply 800 kN of axial compression on each column with each strand stressed to 133 kN. This represented about 25% of the nominal column concentric load capacity. Two additional point loads, 62 kN each, were applied on the beam by means of two prestressing

strands (each stressed to 31 kN), representing the slab loads transferred by two transverse beams. The point loads were applied at 850 mm from the face of each column. The prestressing was done between two HSS (102mm x102mm x 9.5mm) sections, one placed at the top of the beam and the other underneath the foundation. Fig. 4-14(b) shows the gravity loads applied.

Retrofitting the frames required the placement of diagonal strands with a mechanism that would allow progressive engagement. This necessitated access to the underside of the foundation of the test frame, though the top corners of the frame provided convenient locations to place the other ends of the strands. In actual frame buildings this would be done by accessing to the underside of the footing resting on the soil at desired locations, or by providing a separate footing for the cables. In the laboratory, where an existing frame was to be retrofitted and tested, a new foundation had to be built with appropriate allowance for the prestressing strands to support the frame and the cables. The frame was then placed on top of the new foundation. The new foundation was designed to support the frame, including the gravity and lateral load effects. An important aspect of the new foundation was to allow access for the installation of the strands. Two openings were provided and shaped such that they had inclined surfaces perpendicular to the strand directions so that the strands would be stressed against these inclined concrete surfaces when engaged in lateral load resistance. The new foundation was heavily reinforced. It was designed to withstand the anticipated cable forces. Ready mixed concrete with 30 MPa strength was used to cast the new foundation. Fig. 4-16 shows the position of the new foundation relative to the foundation of the existing frame and the laboratory strong floor while illustrating the stages of construction. The new foundation was fixed to the laboratory strong floor by means of 8-75 mm diameter steel bolts to avoid horizontal and vertical movements. Four of these bolts were also used to fix the existing foundation of the frame to the new foundation. The next step in the implementation of the retrofit strategy was to provide holes in concrete at the top beam-column joints, as well as at two corresponding locations in the foundation to connect the cables to have X-bracing. The holes were drilled by a professional concrete drilling/cutting company such that the angle of inclination of the holes would align with the orientation of the strands. The size of the holes was calculated to avoid bearing of the strands

on the sides of the holes during the test when the frame is deformed to a maximum possible lateral drift of 4%. The first test had a single 15 mm diameter 7-wire strands and a 1.0 inch (25 mm) diameter was deemed to satisfy this requirement. Fig. 4-17(a) illustrates the location of one of the holes. The size of the hole was subsequently increased to 2.5 inches (63 mm) for the second tests for which 3-15 mm strands were used along each diagonal. During the drilling of top joints, two longitudinal column bars were cut unintentionally on the right side of the east column. However, this was not expected to affect the test as this region of the column was not critical. None of the column reinforcement was cut in the west column joint located on the actuator side. Two inclined holes were drilled in the original foundation with a 1-1/4 inch (32 mm) diameter bit to pass the strands through for the first test, which was increased to 2.5 in (63 mm) for the second test. Once again, extreme care was exercised to ensure that the cables would not have contact with the concrete edges of the holes to avoid cable damage during the test. Fig. 4-17(b) shows the drilling process for the bottom holes in the existing foundation.

Connections for the strands were designed to secure them on top of the beam-column joints at the top, and at the bottom of the new foundation that was cast specifically to allow for such connections. Different types of strand connections were considered. At the end, it was decided to use threaded cone wedge locker system for fixing the strands. This involved the use of BBR VT CONA CMM Two/Four mono-strand post-tensioning system after some modifications. This post tensioning system locker included wedges, thick wedge retaining plates (anchor heads), thick plates as base plates to support the wedge retaining plates, a flexible galvanized trumpet duck to gather mono-strands in a smaller area with a specified slope, which avoids local stress concentration on strands at sharp edges. 15M (16 mm diameter) spirals were embedded in the concrete of the new foundation to provide reinforcement for the bursting zone. Anchor heads were designed to support a maximum of four high-strength strands under their maximum tensile capacity. They were placed on the support plates. The anchor heads could accommodate strands with a maximum diameter of 15.2 mm. The strands could be locked in using the appropriate size wedges. These cone shape wedges had threaded inside surfaces to grab the strands and lock them in under high tensile forces expected to be developed when the strands fulfill their functions as tension

braces. Fig. 4-18 shows the details of the end anchors. Two L shape plates with triangle stiffeners were designed for each of the two top anchors. They provided support to the anchor head and the plate as shown in Fig. 4-19(a). The size of the hole between the two stiffeners was of the same size as the hole at the tip of the trumpet. This design satisfies the slope needed to avoid local stresses on strands because of the excessive bending expected during the test. They were placed in the outside corner of the beam-column joints, as illustrated in Fig. 4-19(b), with a layer of grout to distribute the load perfectly on the joint surface. The overall schematics of the strand anchorage system is shown in Fig. 4-19(c).

4.4.2 Test set-up and instrumentation

The frames were first secured on the laboratory strong floor with vertical strands applying gravity loads and diagonal PEC strands in place between the diagonal corners with appropriate lengths. Lateral displacement reversals were applied by means of a 1000 kN double acting MTS actuator in deformation control mode. One side of the actuator was connected to a mechano set, consisting of two rigid A-frames and a steel cross beam fixed on the strong floor. The other end of the actuator was attached to one end of the beam by means of a plate assembly. This ensured the application of horizontal force during the push phase. Pulling was done by pulling a plate placed on the other side of the beam, which was attached to the plate at the actuator side by means of 4-25 mm diameter high-strength Dywidag prestressing bars. Fig. 4-20 illustrates the lateral loading system. The frame was subjected to incrementally increasing lateral drift reversals in the plane of the frame. This necessitated the stability of the frame in the transverse direction to simulate the actual frame behaviour in multi-bay multi-story buildings. Therefore, a lateral bracing system was designed to support the frame in the out-of-plane direction.

The frame was instrumented to measure displacements and strains at critical locations. A total of 8 linear variable displacement transducers (LVDT) and 8 cable transducers were used for displacement measurements. Fig. 4-21 shows the locations of all the transducers. In addition to the primary measurements by CT 7 and CT 8 for horizontal frame displacements, the displacement measurements included lateral and vertical movements of foundations and vertical measurements near the column hinging region for anchorage slip

and column rotations. CT5 and CT6 were placed to control the horizontal movement of the original foundation and the new foundation underneath. In addition, a total of 40 strain gauges were placed on reinforcement and prestressing strands.

4.5 Frame tests, data analysis and results

The test frame was retrofitted with two arrangements of PEC strands and tested twice. Arrangement 1 consisted of a single strand in each diagonal, designed to engage in lateral force resistance at 1% drift ratio. Arrangement 2 had multiple strands along the diagonals with one strand engaging at 1.5% drift and the other two at 2% drift ratio.

PEC Arrangement 1 and Test 1:

The first test was for the frame retrofitted with single loose strands placed along two diagonals. The strands were designed and placed on the frame such that each strand would engage in lateral force resistance by developing axial tension at 1% lateral drift in each direction. Each strand was first passed through the opening in the foundation and the pre-drilled hole. It was locked by the wedge in the bottom anchor. Each strand was then pulled from the top corner by a small force, enough to lock the bottom wedge, while also checking for the strand length when it is snug-tight. The strands were then released and locked at their design lengths, which was the length that would allow them to start engaging in lateral force resistance by overcoming the initial slack and starting to stretch. The maximum anticipated lateral frame displacement corresponded to 2.5% drift ratio. Fig. 4-20 shows the frame retrofitted by a single strand in each direction prior to testing.

The frame resistance at 1% drift was 120 kN. After the 1% drift level, the brace provided additional resistance with increased strength and stiffness. The test continued up to 2.5% lateral drifts. The first three deformation cycles were applied at 0.5% drift when the horizontal displacement reached 15 mm. The frame did not develop any new crack during these cycles. The cycles at 1% drift level generated cracks at the bottom 1/3 of the columns and at the ends of the beams at column faces. The deformation reversals at 1.5% and 2.0% drift increased the number of cracks as well as the crack widths. There was no joint cracking observed during these stages of loading. The cycles at 2.5% lateral drift (79 mm lateral

displacement) generated additional flexural cracks. Diagonal cracks were observed on the side faces of columns, as well as at beam column joints. Fig. 4-22 shows the crack patterns at this stage of loading. There was no crushing of concrete observed at this stage of loading. The target drift level was achieved and the test was terminated without inducing significant inelasticity and damage to the frame so that it could be tested once again with the three strands implemented for PEC Arrangement 2 as Test 2.

Experimentally obtained hysteretic lateral force-top horizontal displacement relationship is plotted in Fig. 4-23(a). The relationship clearly shows the intended effect of the retrofit strategy. It initially showed flexible behaviour based on the frame stiffness alone with the strands remaining loose, followed by an increase in stiffness as the strands engaged in force resistance at 1% drift (at 32 mm horizontal displacement and corresponding force resistance of 112 kN). The overall system resistance increased to 288 kN at 2.5% lateral drift.

The frame with PEC Arrangement 1 was modelled and analyzed under the same load reversals using software Seismostruct (Seismosoft 2016) described earlier. The hysteretic response obtained by the numerical analysis is compared with that recorded during the test in Fig. 4-23(b). The comparison shows good agreement between the experimental and numerical responses. Of significance is the proof of the concept developed earlier through analytical research by testing a large-scale reinforced concrete frame with the engagement of the strands at desired level of lateral drift. Further assessment of the comparison indicates slightly softer response of the test frame, which is attributed to the prior cracking of the frame during its previous test. As was indicated earlier, the frame had been tested in an earlier investigation at the University of Ottawa, though the maximum drift imposed was limited to 1.13%, having developed cracking but not yielding of reinforcement. However, the same strength and the overall hysteretic response is in almost perfect agreement with the numerical response. Other factors that may have contributed to the minor differences observed between the experimental and numerical results can be attributed to the frame fixity and the manner in which the gravity loads were applied. The columns were fully fixed in the analysis, whereas they may have experienced some flexibility during the experiment. Furthermore, the gravity load was applied as constant vertical forces in the numerical

simulation. However, during the test, they were applied by means of prestressing strands, which showed variations in stress during loading. The strands on the loading side developed higher forces as they were stretched more, whereas the strands at the other end started unloading themselves, a phenomenon that was not simulated in the numerical analysis. Furthermore, the horizontal components of prestressing strands, opposing the horizontal actuator were not subtracted from the experimental horizontal actuator force, plotted in Fig. 4-23. Although this effect was believed to be small, due to the low levels of axial forces and the limited nature of the lateral drift imposed, it may be a factor explaining some of the minor differences in the hysteretic relationships.

An important aspect of the test program was to validate the performance of the tension braces. Therefore, strain gauges were placed on the strands. The variation in strand stresses is plotted in Fig. 4-24 for each of the two diagonal PEC braces. It can be observed that the strands started developing strains as the applied load approached 112 kN at about 1% lateral drift. The strains in the strands continue increasing until the end of the test with a maximum recorded strain of 0.0039. This level of strain, which is higher than the yield strain of mild steel, is below the yield strain of 1860 MPa prestressing strands. The maximum stress at this stage was recorded to be 780 MPa with a corresponding tension force of 110 kN. Remaining within the elastic range of deformations is one of the benefits of using high strength strands as braces. Elastic response of tension braces promotes self-centering behaviour with reduced permanent deformations in the retrofitted frame.

PEC Arrangement 2 and Test 2:

PEC Arrangement 2 involved testing of the frame with 3 high-strength strands used as tension elements in each direction, where the strands would progressively engage in lateral load resistance. The first strand was designed to engage at 1.5% lateral drift. The remaining two strands were placed with sufficient slack to start developing tension at 2% drift. Fig. 4-25 shows the PEC Arrangement 2 prior to conducting Test 2. Test 2 was performed on the same frame, after replacing the single tension brace in each direction with three 7-wire strands. Having validated the concept with Test 1, the intent was to increase the contribution of lateral load resistance provided by the strands. This time the test was intended to continue

until failure.

The frame initially showed almost the same behaviour as that observed during Test 1 before reaching 2.0% of lateral drift. Cycles at 2.5% generated additional flexural and inclined shear cracks. The presence of higher axial compression in columns, because of the higher diagonal tension forces in the strands, resulted in the onset of concrete cover crushing near the base. The damage observed in the columns, as well as the crack patterns marked on the frame are illustrated in Fig. 4-26. Increase in lateral deformation levels to 3.0% and 3.5% drifts resulted in increased crushing of cover concrete at the ends of the columns. The flexural and diagonal tension cracks in frame elements became wider. The retrofitted frame continued increasing its lateral force resistance because of the contributions of the braces. The damage induced at the end of 3.0% and 3.5% drift cycles are illustrated in Fig. 4-27 and 4-28, respectively. The flexural and diagonal tension cracks in frame elements became wider. The retrofitted frame continued increasing its lateral force resistance during the subsequent deformation cycles because of the contributions of the braces. However, the frame suffered significant damage during 4% drift cycles as shown in Fig. 4-29 and 4-30 could not sustain the third cycle at this deformation level. Significant crushing of concrete occurred in the columns. Furthermore, the buckling of column reinforcement was observed both at the base and at the beam-column joint regions. The frame became unstable in the out-of-plane direction and the test was terminated after two cycles at 4% drift. Fig. 4-30 shows the frame at the end of the test.

Hysteretic response of the retrofitted frame during Test 2 is depicted in Fig. 4-31(a). The frame indicates initially softer response provided by the frame alone, followed by a gradual increase in stiffness and strength as the strands start engaging in force resistance at 1.5 % and 2.0% drift levels. The test exhibited the anticipated behaviour with the frame developing higher lateral force resistance at 4% drift, prior to reaching failure at 588 kN.

The frame with PEC Arrangement 2 was modelled and analyzed under the same load reversals using software Seismostruct (Seismosoft 2016) described earlier. The hysteretic response obtained by the numerical analysis is compared with that recorded during the test in Fig. 4-31(b). The comparison shows reasonably good agreement, with experimental

response indicating stiffer response. The strain gauge data on the strands indicate that the strands were engaged in response about 0.5% drift sooner than what had been intended, resulting in stiffer response sooner than what had been anticipated. Fig. 4-32 illustrates measured forces in strands. As can be seen, the first set of strands were engaged at approximately 30 mm and 40 mm in the push and pull directions, respectively, which correspond to 1% and 1.3% drift ratios. The second set of strands were engaged at lateral displacements of approximately 45 mm and 50 mm, corresponding to 1.5% and 1.7% lateral drift ratios. However, the overall response prediction was deemed to be satisfactory to provide proof of the concept of progressively engaging strands.

The strain data on the strands indicated a maximum strain of 0.0078, which occurred when the frame was pushed to 4.5% and base shear force reached 557kN shortly before the frame became unstable. The maximum strain was recorded in the first strand of the brace between the top east corner and the bottom west corner, developing 1560 MPa stress and 221kN tensile force. The second set of strands (double strands) in the same diagonal direction developed a maximum strain of 0.0071 and the corresponding stress of 1420MPa. The total force in this double-strand brace was 402kN. The braces in the other diagonal direction (between top west corner and bottom east corner) developed similarly high strains, with recorded values of 0.0072 in the first brace and 0.0063 average strain in the double-strand second brace at the maximum lateral drift of 4% with corresponding tensile forces of 204 kN and 358 kN, respectively.

4.6 Conclusions

The following conclusions can be drawn from the combined experimental and analytical investigation conducted for developing progressively engaging cable (PEC) system of seismic retrofitting for non-ductile reinforced concrete frame structures.

- The new tension-only bracing system that was developed for reinforced concrete frame structures, consisting of progressively engaging cables (PEC), demonstrated the desired features of seismic strengthening when needed during response, lateral drift control and improved deformability. Tests of large-scale frames retrofitted with the PEC system, designed to engage in lateral force resistance at higher drift levels proved

the concept of controlled participation in response thereby increasing seismic resistance when needed at higher drift levels without increasing seismic force demands during initial response when the frames alone have sufficient resistance.

- Delayed participation of the PEC system in seismic response results in increased deformability of retrofitted frames. Seismic force resistance of the PEC braces gradually increase as seismic forces are transferred from the decaying response of the non-ductile frame to the brace system, which assumes a higher role in the overall seismic force resisting mechanism of the structure.
- Tightly connected tension-only braces, unlike the PEC system, start resisting seismic forces as soon as the frame is subjected seismic excitations, stiffening the structure, and shortening its period with potentials for increasing seismic force demands. This results in increased axial tension and compression in the attached columns, potentially having detrimental effects at small lateral drift ratios in terms of net column tension, potentials for foundation lift-up and/or column concrete crushing, thereby limiting frame deformability. However, these systems may be effective for increased seismic force resistance and drift control, so long as other detrimental effects are controlled through precautionary measures.
- Multi-cable PEC systems may result in self centering of the frame through elastic response of some of the cables. The use of high-strength 7-wire strands enables the development of elastic response in some of the cable, while other may yield and dissipate energy.

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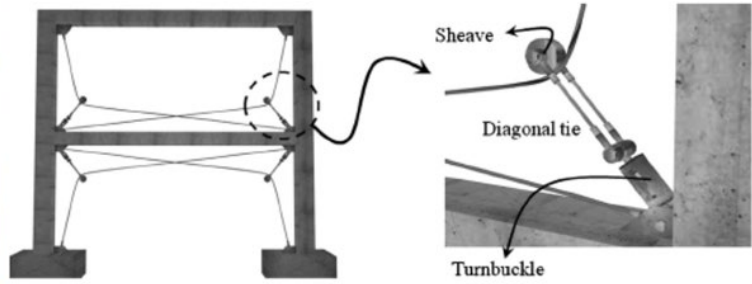
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4.8 Tables and Figures

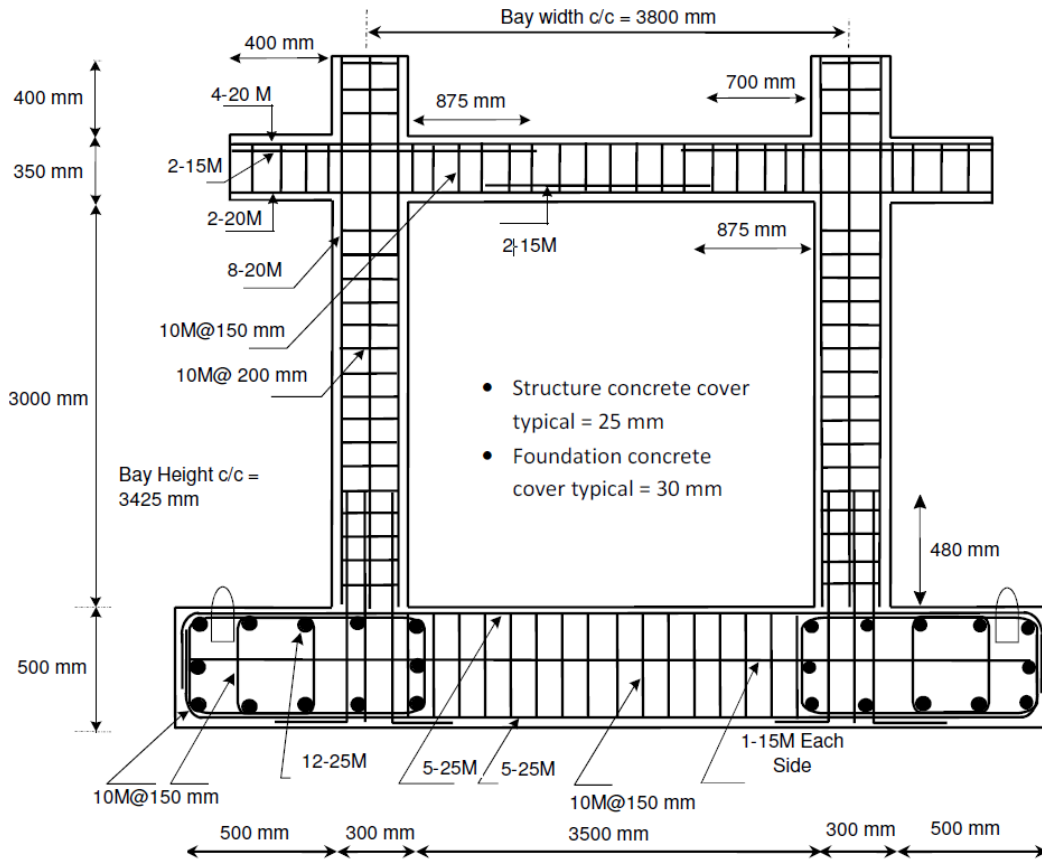


(a) Hou and Tagawa (2009)



(b) Mousavi and Zahrai (2017)

Figure 4-1 Loose cables used for steel frames



(a) Front elevation view

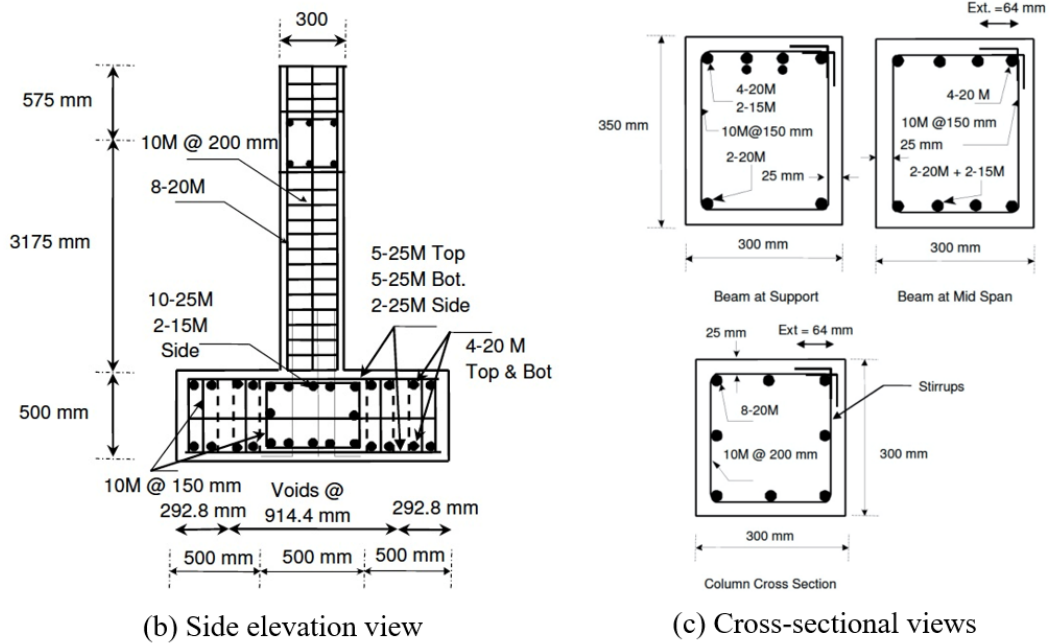


Figure 4-2 Test frame section details

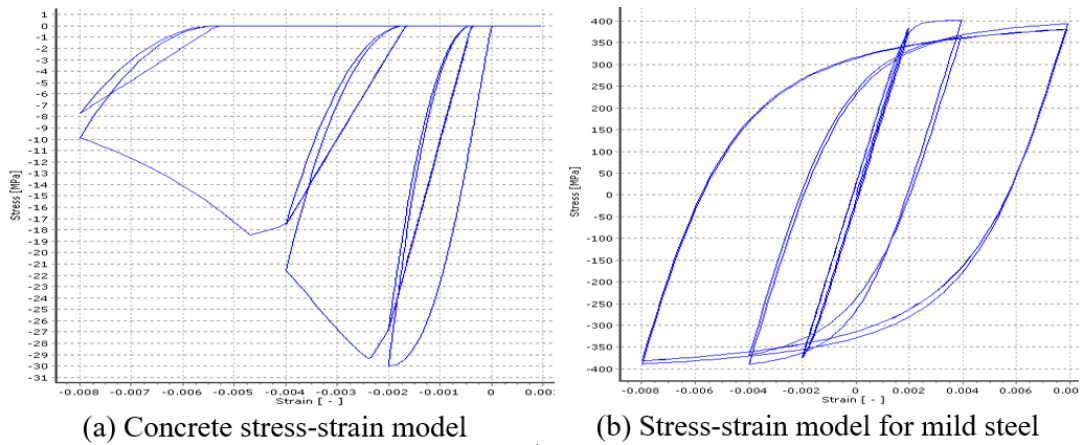


Figure 4-3 Material models for concrete and reinforcing steel

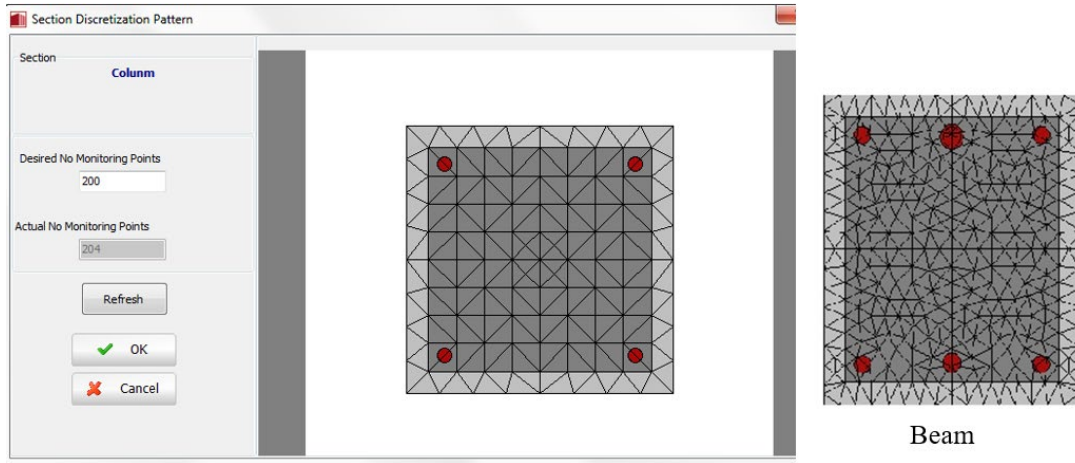


Figure 4-4 Cross-sectional discretization pattern for beams and columns

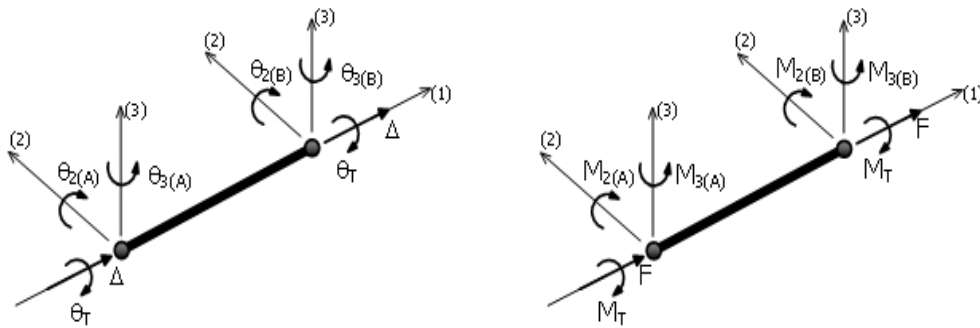


Figure 4-5 Degrees of freedom of both ends of an elements

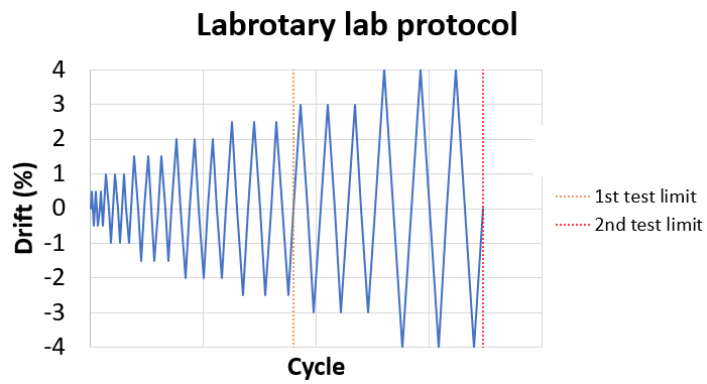


Figure 4-6 Loading protocol used for tests and analyses

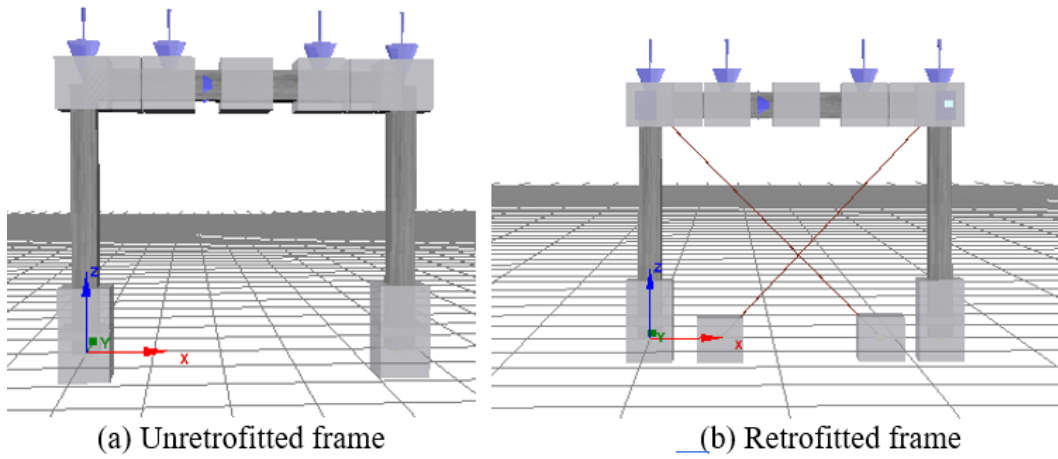


Figure 4-7 Analytical models for the test frames

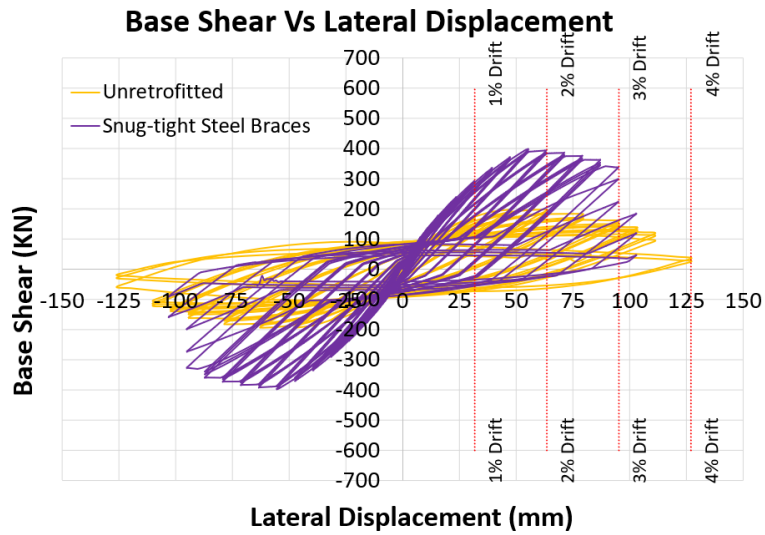


Figure 4-8 Hysteretic relationships for unretrofitted and retrofitted frames

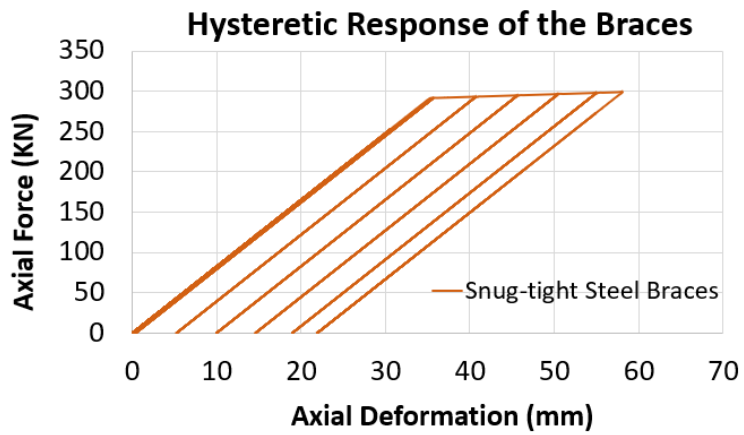


Figure 4-9 Hysteretic relationship of a snug-tight steel brace

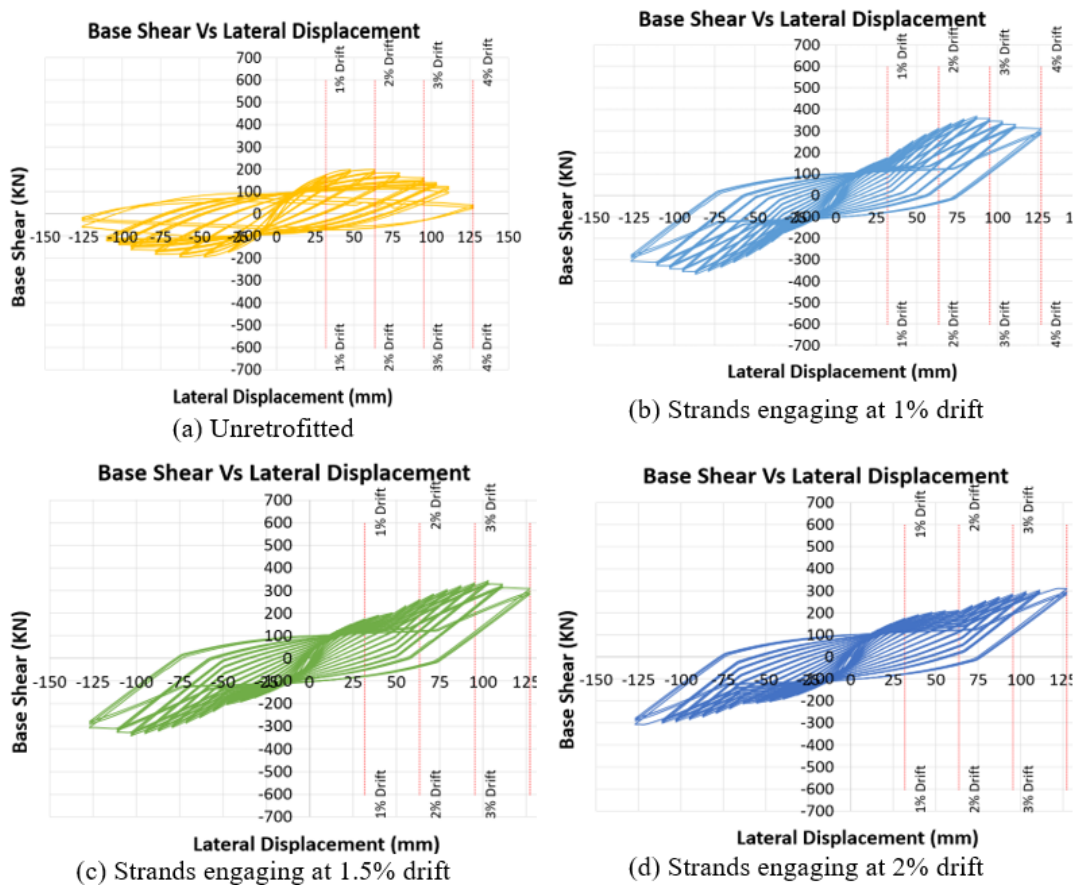


Figure 4-10 Hysteretic relationships of buildings with strands engaging at different drift levels

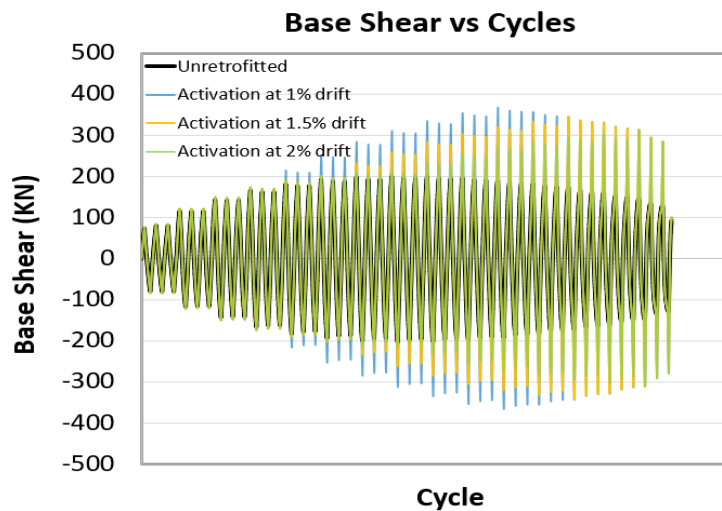


Figure 4-11 Distribution of total base shear between the frame and the tension braces under progressively increasing lateral deformation cycles

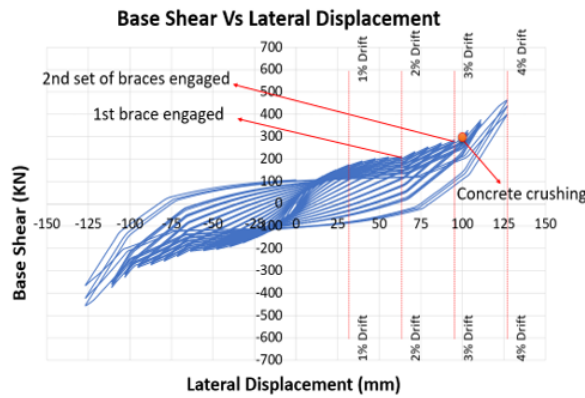
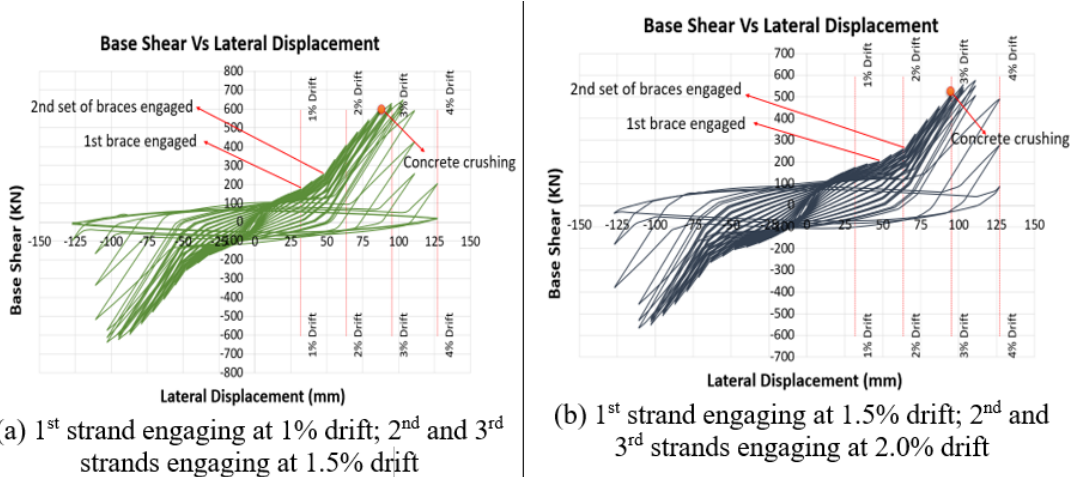
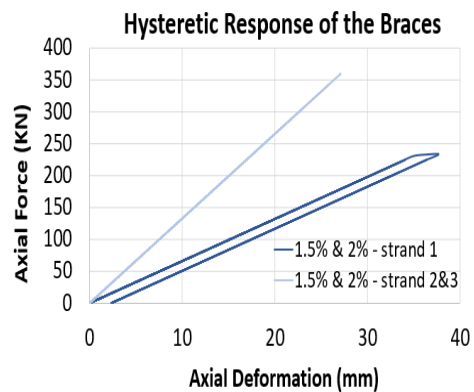
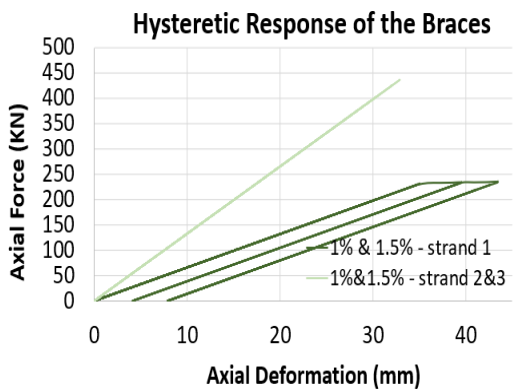
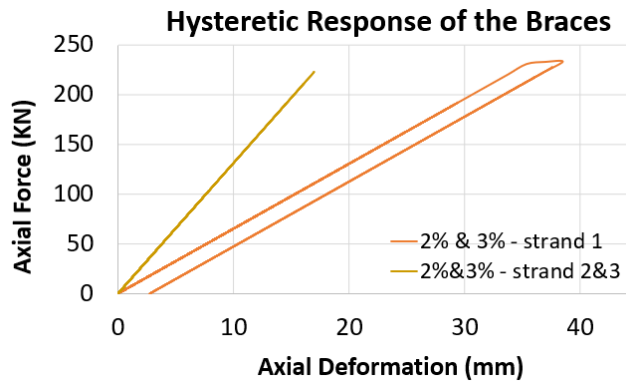


Figure 4-12 Hysteretic relationships of buildings with multiple strands engaging at different drift levels



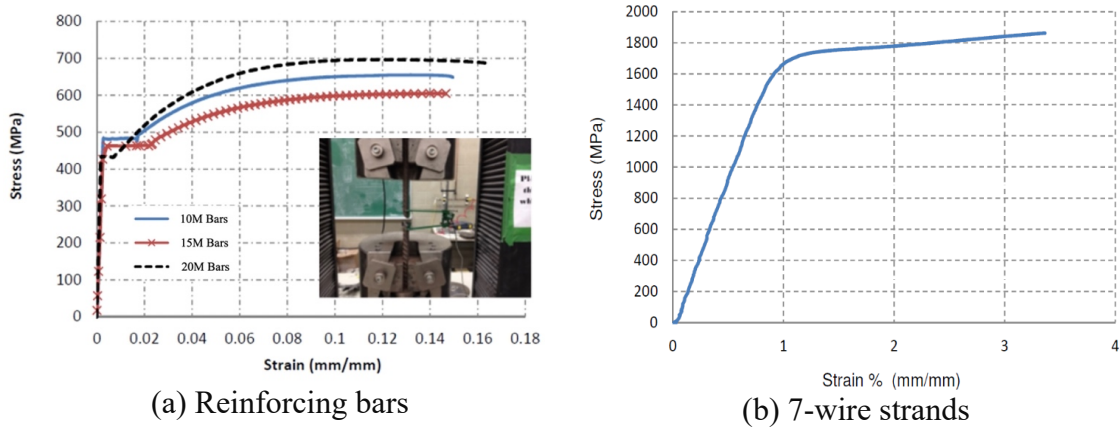
& 3 engaged at 1.5% drift

and 3 engaged at 2% drift



(c) Strand 1 engaged at 2% drift; Strands 2 and 3 engaged at 3.0% drift

Figure 4-13 Hysteretic relationships of buildings with multiple strands engaging at different drift levels



(a) Reinforcing bars (b) 7-wire strands

Figure 4-14 Stress-strain relationship for steel used in the test frame

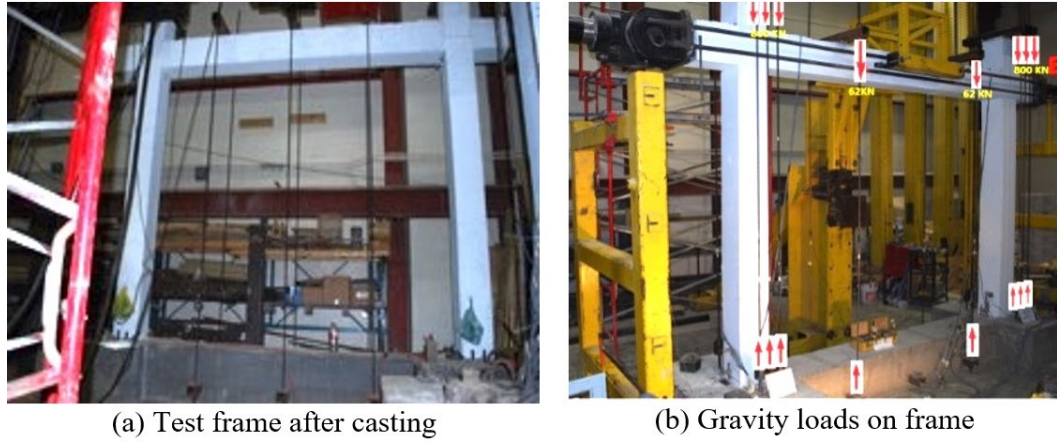


Figure 4-15 Test frame

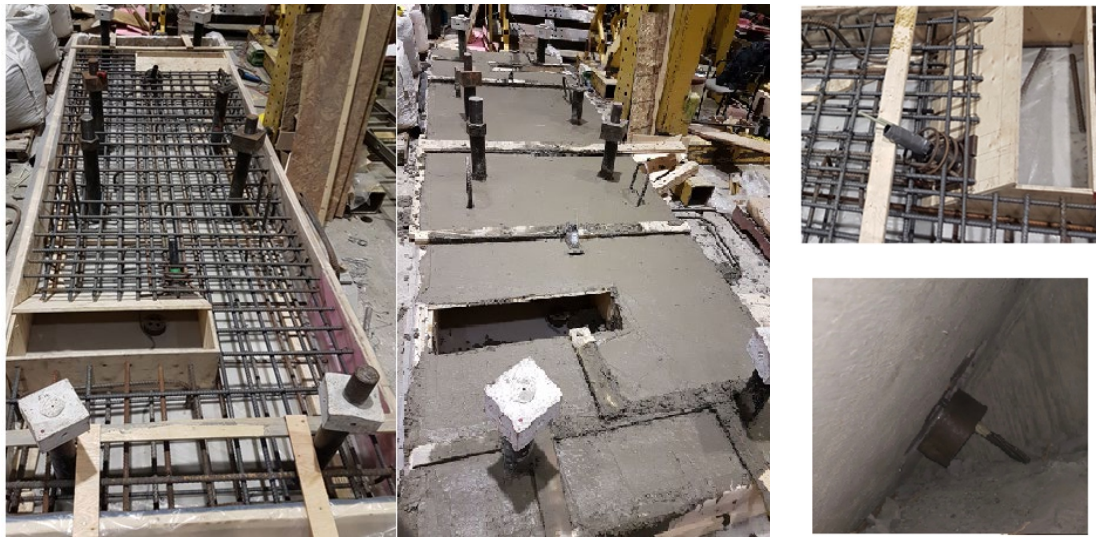
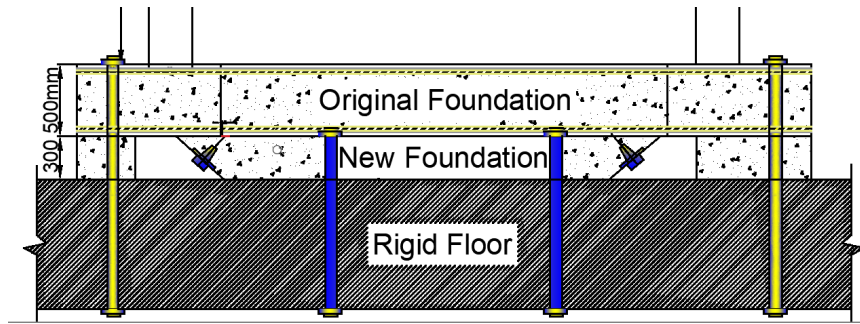
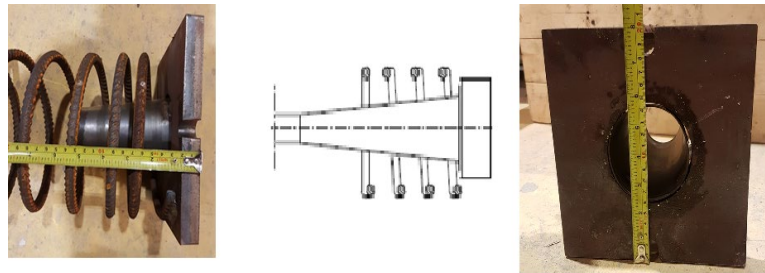


Figure 4-16 Preparation of the new foundation to facilitate the placement of diagonal strands



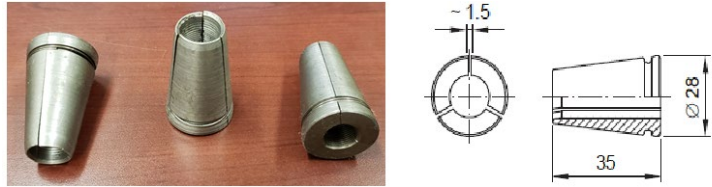
Figure 4-17 Preparation of the new foundation to facilitate the placement of diagonal strands



(a) Spiral transverse reinforcement, trumpet and support plate

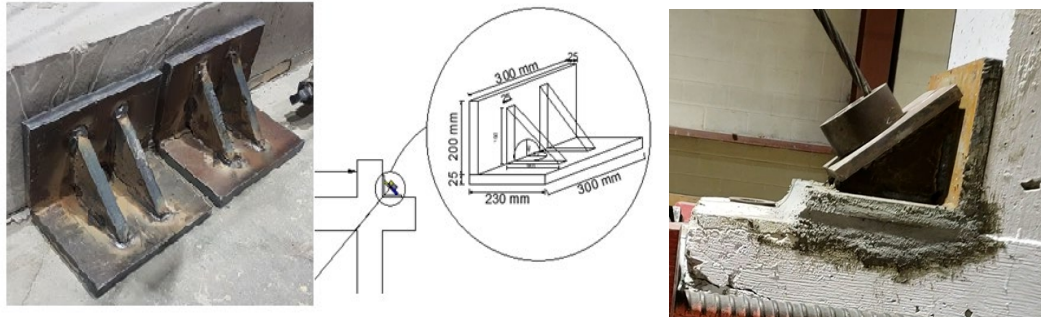


Anchor head front Anchor head back Anchor head side
 (b) Anchor head that can accommodate up to 4 strands



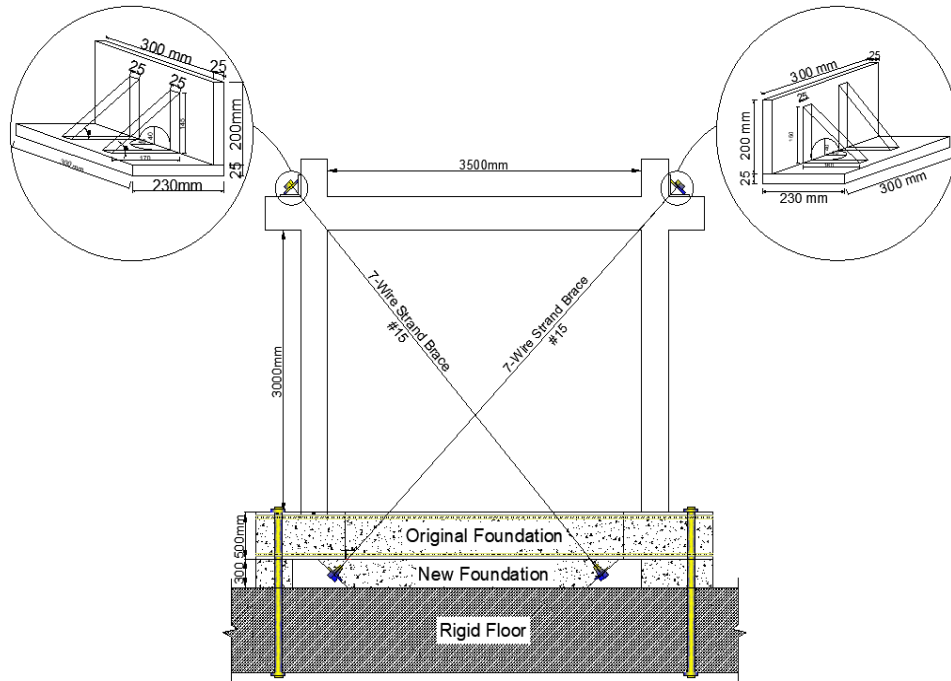
(c) Threaded cone shaped wedges to fix seven wire strands

Figure 4-18 Anchorage details for diagonal PEC strands



(a) Top corner L-shaped bearing plates

(b) Grouting the surface



(c) Schematic of retrofitted frame showing strand anchors

Figure 4-19 Top bearing plates and anchorage locations



Figure 4-20 Test setup

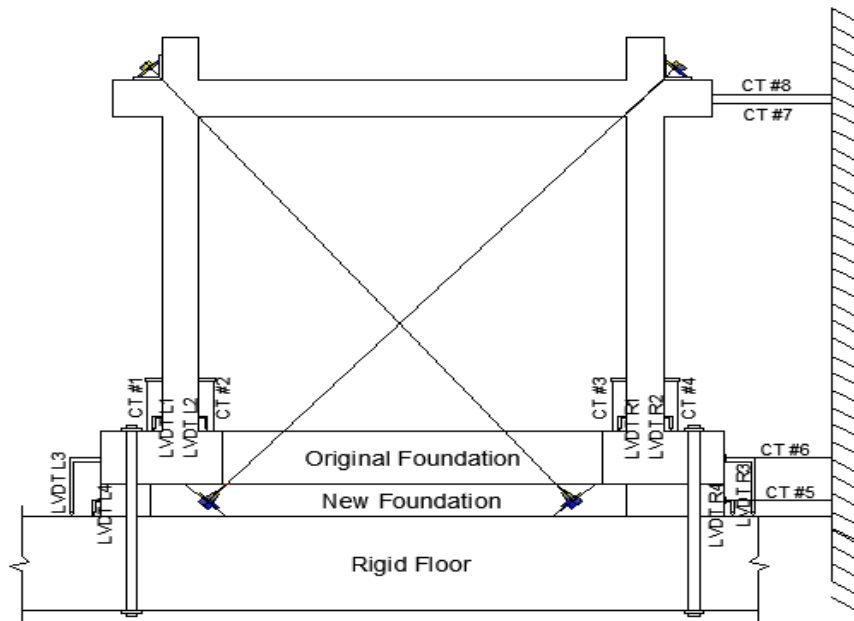


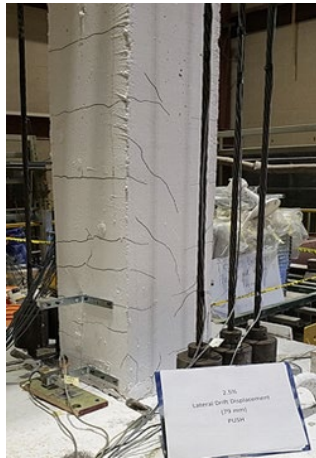
Figure 4-21 Instrumentation



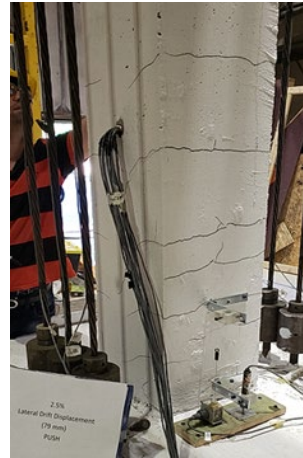
(a) West beam-column joint



(b) East beam-column joint

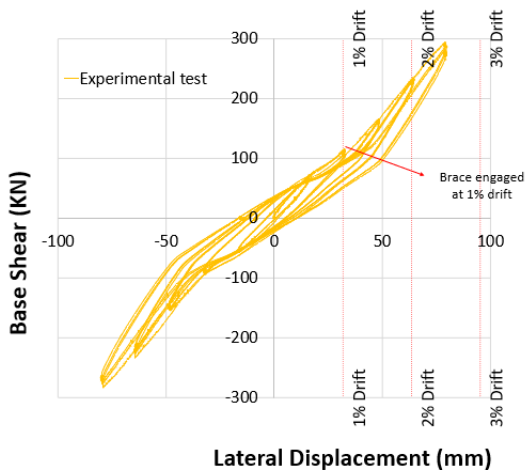


(c) West column

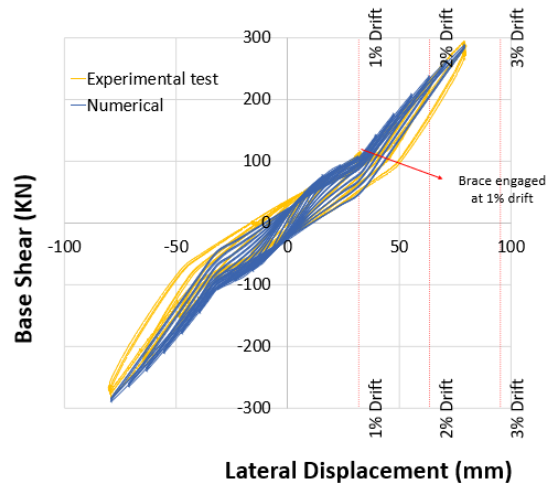


(d) East column

Figure 4-22 Crack patterns at the end of the test after completing 2.5% lateral drift cycles

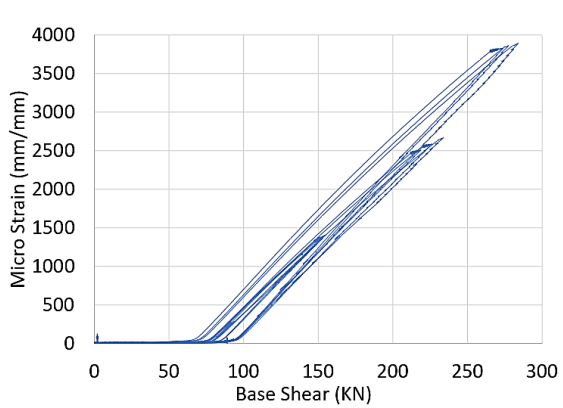


(a) Experimental results

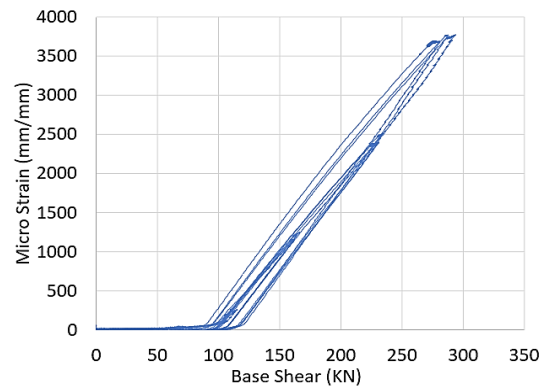


(b) Experimental versus numerical results

Figure 4-23 Comparisons of hysteretic relationships for Test 1 with PEC Arrangement 1



(a) Strand 1: bottom west to top north corner



(b) Strand 2: bottom east to top west corner

Figure 4-24 Variation of forces in strands during the first test with PEC Arrangement 1

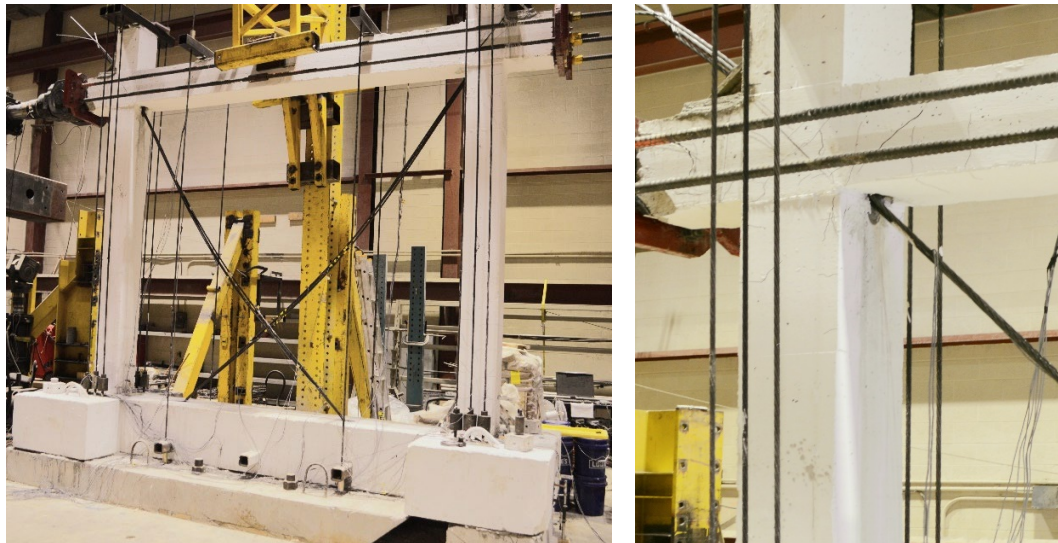


Figure 4-25 PEC Arrangement 2



(a) East column

(b) West column

(c) East top joint

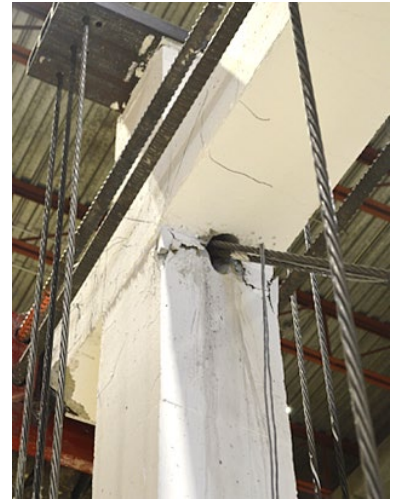
Figure 4-26 Damage observed at 2.5% drift - PEC Arrangement 2



(a) West column



(b) East column

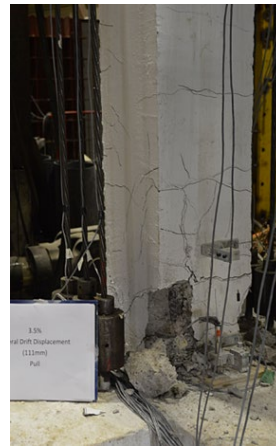


(c) West top joint

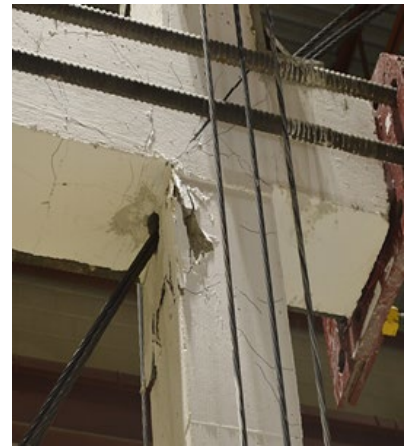
Figure 4-27 Damage observed at 3.0% drift - PEC Arrangement 2



(a) West column

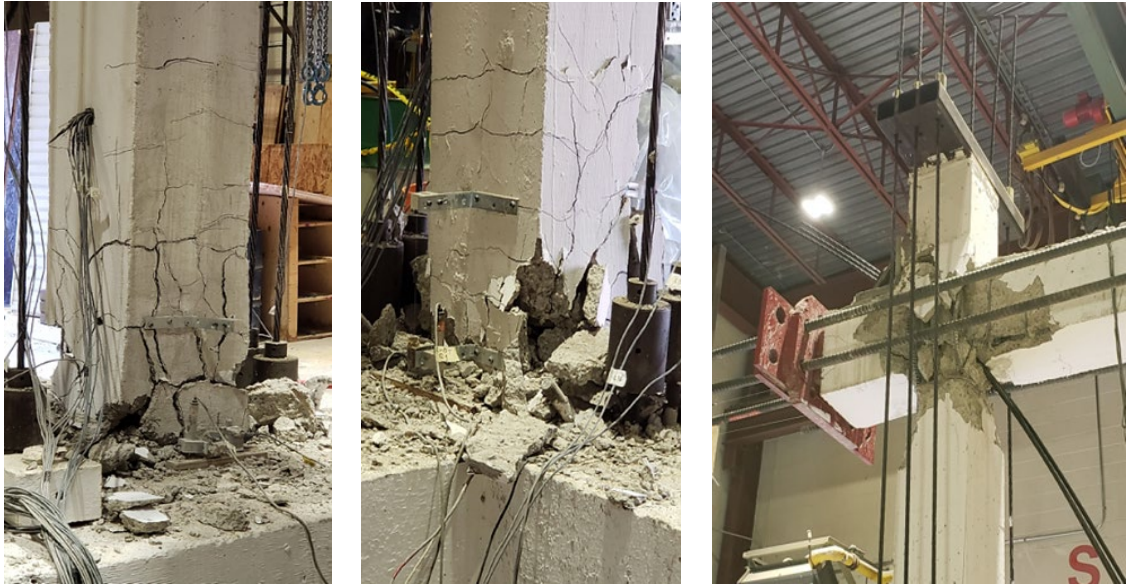


(b) East column



(c) West top joint

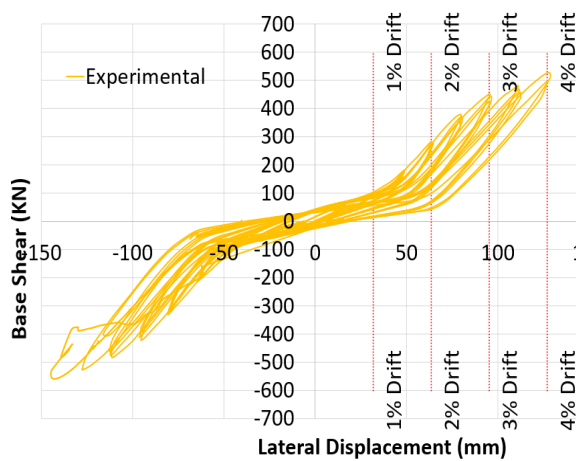
Figure 4-28 Damage observed at 3.5% drift - PEC Arrangement 2



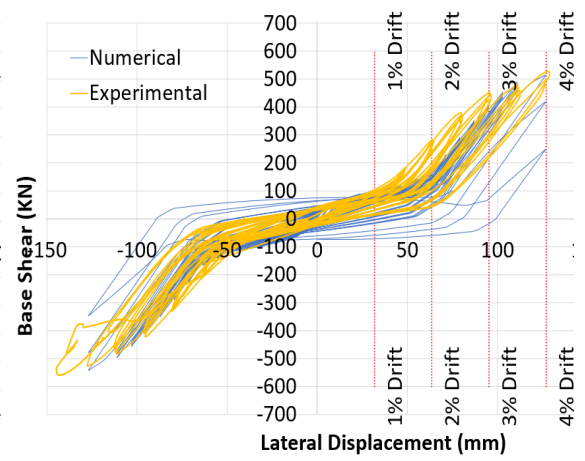
(a) West column (b) East column (c) East top joint
Figure 4-29 Damage observed at 4.0% drift - PEC Arrangement 2



Figure 4-30 Test frame at the end of the test - PEC Arrangement 2

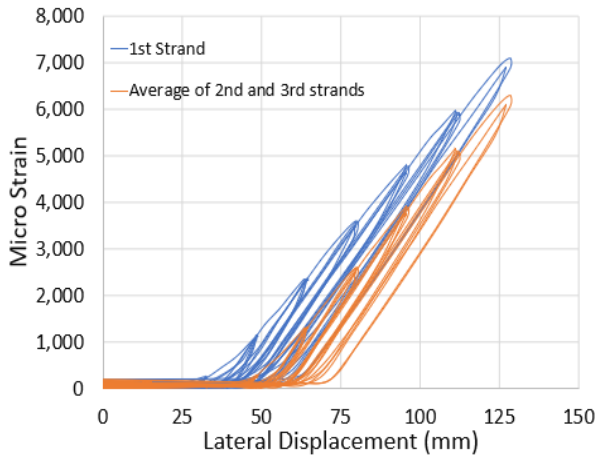


(a) Experimental results

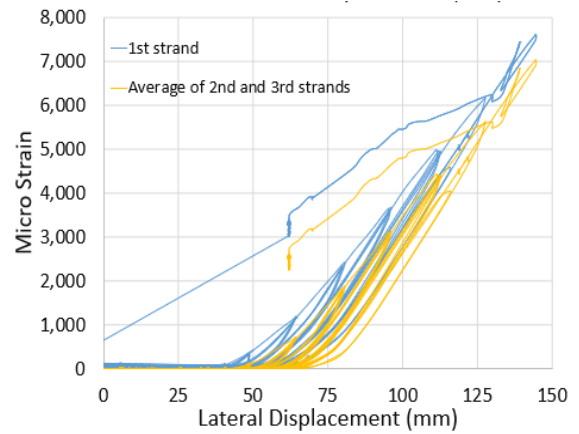


(b) Experimental versus numerical results

Figure 4-31 Comparisons of hysteretic relationships for Test 2 with PEC Arrangement 2



(a) Strand 1: bottom west to top north corner



(b) Strand 2: bottom east to top west corner

Figure 4-32 Variation of forces in strands during the second test with PEC Arrangement

2

CHAPTER 5

5 Development of Progressively Engaging Tension Braces for Seismic Retrofit of Reinforced Concrete Frames

Abstract:

Reinforced concrete buildings built prior to the enactment of modern seismic codes often have deficient strength and/or ductility to withstand strong earthquakes. A common seismic risk mitigation strategy is to implement seismic retrofits to improve their performance. For non-ductile frame buildings, conventional retrofit techniques include lateral bracing by adding reinforced concrete shear walls or structural steel bracing systems to increase strength and stiffness, while also aiming for improved deformability and drift control. After the 1985 Mexico City Earthquake, tension only cable braces have also been used and investigated as a seismic retrofit technique. In this approach, high-strength prestressing strands or cables are placed diagonally for lateral bracing.

Lateral bracing of seismically deficient frames results in increased stiffness, reduced building period and increased seismic force demands. They also impose higher force demands in the attached columns and building foundation, increasing column axial tension and compression, potentially leading to foundation uplift and/or concrete crushing as they improve seismic shear resistance of the overall system. A new tension-only cable bracing system has been developed as part of the current research program through experimental and analytical research (Khosravi and Saatcioglu (2021)a), consisting of progressively engaging cable (PEC) system whereby high-strength cables are engaged in seismic force resistance in a controlled manner. In this system, the strands are placed loosely as tension elements to engage in force resistance progressively, elimination increases in seismic force demands during more frequent earthquakes of low to medium intensity and engaging in frame bracing when the earthquake intensity is increased. In this application, the strands initially remain unstressed until the lateral drift reaches a critical value, beyond which the frame requires additional lateral resistance. The application of the new system is demonstrated by selecting a medium-rise and a low-rise non-ductile reinforced concrete frame buildings as representatives of pre-1970s design practice, located in a region of high seismicity. The buildings are retrofitted with either snug-tight tension cable braces or with the PEC system that engage at different levels of pre-determined drift levels. The results are

compared in terms of hysteretic responses, indicating seismic force and deformation demands. The superior performance of the PEC system in terms of improved seismic force and deformation resistance is demonstrated.

Keyword: Concrete frames, bracing, cables, ductility, drift control, earthquake engineering, reinforced concrete, seismic design, seismic retrofit.

5.1 Introduction

Performance of buildings during past earthquakes has demonstrated poor performance of non-ductile reinforced concrete frame building when subjected to strong earthquakes (Saatcioglu et al. 2013, Saatcioglu et al. 2006, Mitchell et al. 1995). Because it is not economically feasible to replace these buildings with new buildings designed using strength and ductility requirements of current building codes, seismic retrofit remains to be a viable seismic risk mitigation strategy. Seismic retrofit of existing frame buildings is implemented either at the element level, as retrofitting of columns, beams, beam-column joints or walls; or globally affecting the entire lateral load resisting system either by adding lateral bracing for strength, stiffness and drift control, or by changing the dynamic characteristics of buildings through the use of supplementary damping or base isolation devices. Element level seismic retrofit techniques are intended to improve seismic force-deformation characteristics of individual elements or components. System level seismic retrofit techniques are implemented to affect the global structural behaviour of the lateral load resisting system. Global retrofit techniques often involve introducing lateral braces to the structure or enhancing the existing structural or non-structural bracing elements to improve their behaviour under seismic excitations. Conventional bracing systems involve adding new concrete structural walls (e.g. Altin et al. (1992), Arslan et al. (2010), (Canbay et al. 2003), (Frosch et al. 1996)) or structural steel braces (e.g. Badoux and Jirsa (1990), Bartera and Giacchetti (2004), Bush et al. (1991), Fukuyama and Sugano (2000)). Additional bracing systems have been developed in recent years in the form of enhancing existing non-structural infill walls with fibre-reinforced polymer composites (FRP) (e.g. Arifuzzaman and Saatcioglu (2012), Saatcioglu (2019)) and using buckling restrained braces (e.g. Tremblay et al. 1999, 2006, 2011; Uang and Nakashima (2004), López et al. (2004), and

Al-Sadoon et al. (2020)). The bracing systems also include compression-only (Caron (2010)) and tension-only braces (Pincheira José and Jirsa James (1995), Shalouf and Saatcioglu (2006), Carrière (2008), Molaei and Saatcioglu (2014)).

Previous research demonstrated the effectiveness of structural bracing systems. They provide retrofit solutions to existing non-ductile frames, which typically have lower lateral force capacities. These systems have different degrees of interference with existing buildings, some highly intrusive, affecting the cost of construction as well as the building down time. Lateral bracing results in increased stiffness, reduced building period and increased seismic shear demands. They also impose higher force demands in the attached columns and building foundations, increasing column axial tension, potentially leading to foundation uplift, while they improve seismic shear resistance of the overall system. A new tension-only cable bracing system has been developed as part of the current research program through experimental and analytical research ((Khosravi and Saatcioglu (2021)a), comprising of progressively engaging cables (PEC). The system consists of high-strength cables/strands, developing seismic force resistance in a controlled manner, each cable becoming active at a different level of lateral drift. In this system, the strands are placed loosely as tension elements to engage in force resistance progressively, without increasing seismic force demands during more frequent earthquakes of low to medium intensity and engaging in frame bracing when the earthquake intensity is increased, becoming a threat to building safety. The strands initially remain unstressed until the lateral drift reaches a critical value, beyond which the frame requires additional lateral resistance to be provided by the tension cables. The system has been validated by testing large-scale reinforced concrete frames under simulated seismic loading. Fig. 5-1 shows two frame tests with initially loose strands. The frame during Test 1 had single strands in each direction, designed to engage in lateral force resistance at a pre-determined drift ratio of 1%. Fig. 5-1(b) shows the hysteretic response obtained during the test, exhibiting initially softer response, followed by increased stiffness beyond the 1% drift level upon the engagement of the strands in lateral load resistance. The frame during Test 2 was equipped with three strands in each diagonal, designed such that the first strand would begin resisting lateral forces at 1.5% drift and the remaining two strands would begin resisting lateral loads at 2% lateral drift. Fig. 5-1(d)

shows the hysteretic response obtained during the test, once again showing the participation of strands gradually at pre-established drift levels. An analytical study was also conducted on the test frame with three different PEC strands subjected to the same incrementally increasing later deformation reversals employed in the experimental research. The analysis was conducted using software Seismostruct (Seismosoft 2016) with the analytical model shown in Fig. 5-2. These analyses consisted of three scenarios with three cables in each case ((Khosravi and Saatcioglu (2021)a). The first scenario has the first cable activated at 1% drift and the other two cables activated at 1.5%. The second scenario has the first cable activated at 1.5% and the other two activated at 2.0% drift, and finally the third scenario has the first cable activated at 2% and the other two activated at 3.0%. The hysteretic relationships obtained are shown in Fig. 5-3 with the slopes of hysteresis loop becoming stiffer after the engagement of each cable. This provides additional results for the proof of concept of PEC cables.

The concept of delayed participation of tension braces were also developed by Hou and Tagawa (2009), and Mousavi and Zahrai (2017) for steel frames. The former group of researchers used a cylindrical steel element in a central location of the X arrangement of tension cables, which allowed an initial slack in the system, delaying the development of tension forces in cables. The latter group used additional steel extenders tied to the cables at frame corners to leave slackness in the cables, equipped with turnbuckles for the adjustment of the cable slackness. Both groups of researchers applied the concept to scaled structural steel test frames and demonstrated their effectiveness.

The objective of the current phase of research is to demonstrate the application of the PEC system of seismic retrofit to two prototype buildings located in a high seismic region of Canada. The buildings consist of a medium-rise and a low-rise non-ductile reinforced concrete frame building, representative of pre-1970s design practices. The buildings are retrofitted with snug-tight tension cable braces and PEC system that engages in lateral force resistance at different levels of pre-determined drift levels. The results are compared with those obtained from the analysis of unretrofitted buildings to illustrate the significance of the seismic retrofit strategies employed.

5.2 Selection of prototype buildings and dynamic analysis

Two reinforced concrete frame buildings with two different heights were selected for seismic retrofitting to demonstrate the effectiveness of the PEC system. The buildings either had a 6-storey or a 2-storey height, representing mid-rise and low-rise buildings. The floor plan was selected as a modified version of the frame building designed in the Canadian Concrete Design Handbook (CAC 2008). Fig. 5-4(a) illustrates the floor plan, which was common to both buildings. The buildings were designed for Vancouver, Canada, using the 1965 edition of the National Building Code of Canada NBC ((NRCC 1965)). The design internal forces were governed through the analysis of the frames carried out by using ETABS software ((CSI 2015)), and the frames were designed according to ACI (1963). Fig. 5-4(b) and Fig. 5-4(c) show the computer models used for design. The buildings were designed for gravity loads that consisted of dead and live loads, and lateral loads that consisted of wind and seismic loads. The gravity loads included superimposed dead loads of 1.0 kPa for partitions, 1.0 kPa for roofing and mechanical, and 1.6 kPa mechanical in the middle 5.7 m span. The design live loads included 4.9 kPa for the first floor and 2.4 kPa for the other floors with 1.3 kPa snow load on the roof, 25% of which was included in the seismic mass. The wind load was selected as 1.05 kPa and seismic forces were calculated as per the 1965 NBC ((NRCC 1965)). Concrete compressive strength of $f'_c = 30$ MPa, and reinforcement yield strength of $f_y = 400$ MPa were used in both building designs. This resulted in 450 mm square exterior columns and 500 mm square interior columns with 400 mm x 600mm beams for the 6-storey building; and 400 mm square columns and 400 mm x 500 mm beams for the 2-storey building. The fundamental periods of the 6-storey and 2-storey buildings were 1.65 sec and 0.84 sec, respectively, based on cracked section stiffnesses (60% and 40% of gross section properties for columns and beams) and the empirical code expressions given in the (NBCC 2015).

The buildings were analyzed using “Seismostruct” software (Seismosoft (2015)), which is a powerful finite element tool for non-linear dynamic response history analysis. The software has element models for beams and columns with capabilities for geometric and material non-linearities. The flexural behaviour of beam and column sections is obtained

through the integration of stresses and strains in individual fibers with nonlinear uniaxial material models assigned to each fiber. The member response is obtained by fully accounting for the spread of inelasticity along the member length by integrating individual sectional response. This type of modelling is considered to be an accurate representation for flexural behaviour since it can consider inelastic behaviour along the entire length of a structural member. It is recommended to have at least 200 fibers in each cross section (Seismosoft 2015). In addition to the section fibers, up to 7 integration sections may be needed to accurately model the hardening response of members, following 4 or 5 integration sections recommended by Seismostruct manual to ensure response in the softening range. Seismostruct also considers plastic-hinging within a fixed length of the element ((Scott and Fenves 2006)). The modelling technique used requires the specification of the potential plastic hinge length. The plastic hinge length was calculated according to Paulay and Priestley (1992) formulation. The buildings and the materials were modelled as described in the next section.

Frame deformations were calculated considering inelastic behaviour of elements. P-Delta effects were incorporated into the analysis by calculating large displacements through the total chord rotation formulation developed and implemented by (Correia and Virtuoso 2006). The chord rotation formulation is based on an exact description of the kinematic transformations associated with large displacements and three-dimensional rotations of beam-column members. This leads to the correct definition of element deformations and forces, as well as to the effects of geometrical non-linearities on the stiffness matrix. The implementation of this formulation considers small deformations relative to the element's chord, notwithstanding the presence of large nodal displacements and rotations. In the local chord system of the beam-column element, six basic displacement degrees-of-freedom and corresponding element internal forces are defined, as shown in Fig. 5-5. The internal forces developed by inelastic frame elements are computed by the software. These values are correlated with axial forces, shear forces and moments (caused by flexural or torsional) at the end of each node. By finding both forces and deformations (rotations) at the ends of each element, the hysteretic behaviour can be obtained in the form of a force-displacement relationship.

Dynamic analysis was conducted using 12 different seismic records generated for western Canada (Atkinson 2009). The records consisted of 4 short records (S1-S4) with a maximum PGA of 0.44g, 4 long records (L1-L4) with a maximum PGA equal to 0.4g, and 4 Cascadia records (C1-C4) with a maximum PGA of 0.25g. The time histories of the records are shown in Fig. 5-6. The comparisons of response spectra with the design spectrum The comparisons of response spectra with the design spectrum by using “SeismoMatch” program (Seismosoft 2016) are shown in Fig. 5-7. The damping considered in all the analyses was 5% of critical damping. The analytical models shown in Fig. 5-4 were used to conduct dynamic analyses. The floor slabs were considered as rigid diaphragms and the columns were considered to be fully restrained at the foundation level against all rotations and displacements in all directions. All the structural members were modelled using inelastic material models for concrete and reinforcing steel. The concrete model used was developed by (Madas 1993) following the stress strain relationship for confined concrete proposed by Mander et al. (1988). The cyclic behaviour of concrete as proposed by (Martínez-Rueda and Elnashai 1997) as illustrated in Fig. 5-8(a) was adopted for all frame elements. The reinforcing steel was modelled using a uniaxial stress-strain model for mild steel. It was based on the stress-strain relationship proposed by (Menegotto and Pinto 1973), followed by the isotropic hardening rules proposed by (Filippou et al. 1983). The cyclic seismic loading rules were recommended by (Fragiadakis et al. 2008). The model is illustrated in Fig. 5-8(b). It was indicated to be suitable for modeling the behavior of reinforced concrete elements under reversed cyclic loading, especially for buildings subjected to seismic loading where significant load reversals might occur.

5.3 Un-retrofitted buildings

The un-retrofitted frame buildings were subjected to the 12 different seismic records indicated in the preceding section. The results indicate that the 6-storey un-retrofitted building exceeded the code specified storey drift limit of 2.5% (NRCC 2015) and reached collapse by exceeding the crushing limit of concrete and becoming unstable after 5% lateral drift under the Cascadia seismic record, CW3. This is consistent with the response spectra presented in Fig. 5-7, in which the spectral values for the 6-storey building with initial

fundamental period of 1.65 sec show the highest response accelerations for Record CW3. The corresponding maximum base shear force for the building was 7226 kN. The first yielding in longitudinal column reinforcement occurred at 0.8% of the first-storey drift at 5150 kN of base shear force. The crushing of cover concrete occurred at 1.33% lateral drift and the crushing of column core took place at 2.5% storey drift. The 2-storey building also developed highest response under the same earthquake record (Cascadia – CW3) with a maximum inter-storey drift of 3.69% and a maximum base shear of 3029 kN. These response quantities are excessive for non-ductile older buildings and indicate high level of damage. Table 5-1 provides a summary of analysis results for both mid-rise and low-rise un-retrofitted buildings. Fig. 5-8 shows the first-storey base shear versus lateral drift hysteretic relationships.

5.4 Retrofitted buildings

The two prototype buildings selected were retrofitted with tension braces, consisting of 1860 MPa high-strength prestressing strands. Two types of steel cables were used for two types of tension braces, where each type was implemented in both buildings. Fig. 5-9 illustrates the models used for the analysis of buildings. The 6-storey building was retrofitted by bracing 4 bays in the critical short direction and the 2-storey building had two braces because of the lower seismic base shear observed in this building. The strands were modelled as tension elements with stress-strain relationships as established through coupon tests of the strands used, as illustrated in Fig. 5-10. They were attached to the joints through gap elements. The gap links were defined in a way that the strands worked only in tension and no compression force was transferred to the braces. The first retrofit strategy involved the use of snug-tight strands with the length of each strand set to fit the diagonals between the lower and upper opposite corners of the selected bays, without any prestressing. The number and the size of the strands were established through preliminary analyses of 2-dimensional plane frames to save computational time. The frames were subjected to the Cascadia CW3 record, which was found earlier to be critical for the unretrofitted buildings. The criterion used to establish the brace size was to reduce the maximum inter-storey drift to a level lower than the upper limit of 2.5% indicated in the 2015 NBC (NRCC 2015). This

resulted in the use of three different size cables for the 6-storey building, consisting of bundled or multiple strands to form steel areas that correspond to cables having a diameter of; i) 56 mm for the first floor, ii) 40 mm for the second and third floors, and iii) 10 mm for the upper floors. The 2-storey building was retrofitted with two 7-wire strands having a total nominal diameter of 30 mm for the first floor and a single strand with a nominal diameter of 15mm for the second floor.

The second retrofit strategy comprised of progressively engaging, initially loose strands (PEC System). Since the most critical floor level was established to be the first floor in the response of the un-retrofitted building, the strands for the first floor were designed to have multi-stage of engagement in seismic resistance. Accordingly, two bundles of strands were used as tension braces for the first storey; i) strands with an equivalent nominal diameter of 35 mm to engage in force resistance at 1.5% inter-storey drift, and ii) strands with an equivalent nominal diameter of 40 mm to engage in seismic response at 2.0 % inter-storey drift level. Second and third stories were braced with strands having an equivalent nominal diameter of 40 mm, while the fourth level braces had a nominal diameter of 20 mm. The top two stories, i.e., the fifth and sixth floor levels had strands having a nominal diameter of 15 mm. The braces above the first-floor level had a single level of engagement in response, design to take forces when the inter-storey drift ration exceeded 1%. For the two-storey low-rise building, the braces for the first and the second floors had a nominal diameter of 30 mm and 15 mm, respectively, both designed to engage in seismic response at 1% inter-storey drift.

The 3-D models of both retrofitted buildings were developed and analyzed under the 12 earthquake records selected for the region. The results are summarized in Table 5-2 and indicate that, just like the companion unretrofitted buildings the results of which are summarized in Table 5-1, the highest response was obtained from the Cascadia CW3 record. The maximum base shear under this record resulted in 15,363 kN when snug-tight cables were used in the 6-storey building. The use of PEC system resulted in about 25% reduction in base shear to 11,749 kN. Furthermore, the maximum axial tension in the snug-tight first-storey cables was recorded to be 2,839 kN, corresponding to a maximum tensile strain of

0.0057. The maximum tension in PEC cables was 1,799 kN, representing a 36% reduction in cable tension. Similar observations were made for the 2-storey buildings. The snug-tight cables resulted in a maximum base shear of 5,599 kN, as opposed to the PEC system, which led to a maximum base shear of 4,919 kN. The maximum axial tension in the first-storey braces of the 2-storey building reduced from 1,301 kN for the snug-tight retrofit scheme to 904 kN for the PEC system. Fig. 5-11 shows the cable forces developed at the first-storey level. The reduction in bracing forces resulted in reduced axial forces in the critical first storey columns to which the braces were attached, eliminating uplift in columns due to excessive tension and the crushing of column concrete under high axial compression. The time histories of the first-storey column (Column C1) shear and axial force demands are illustrated in Fig. 5-12. The use of snug-tight cables in the 6-storey building resulted in very high column shear force demands in excess of 1500 kN during the first 53 sec. of response as opposed to the PEC system which showed about 200 kN column shear demand. This is the range of response when displacements were relatively low, as would be in the case of low to moderate seismic excitations. At maximum response, just prior to reaching 55 sec. in response, the snug-tight cables induced 2,648 kN of shear force on Column C1, whereas the maximum shear response due to the use of the PEC system was 1,618 kN. Corresponding reductions in column axial tension was observed; 2839 kN in Column C1 when snug-tight cables were used down to 165 kN in the PEC system. Similar reductions in column axial tension was observed in the retrofitted 2-storey buildings. The snug-tight cables generated 646 kN of axial tension at bottom of Column C1 while in the same force was reduced to 326 kN when the PEC system was used. The maximum response quantities for both buildings are summarized in Table 5-2.

The hysteretic response of the critical first storey of all the buildings analyzed are compared in Fig. 5-13. The results indicate that the retrofitted buildings have high seismic force resistances. The snug-tight cables led to more rigid buildings with reduced fundamental periods and increased seismic force demands. The cables remained elastic, and the building responded rigidly as soon as the seismic excitation has begun with reduced deformation demands and increased force demands. The PEC system engaged in seismic resistance gradually. In the case of 6-storey building, the first set of cables were engaged at

1.5% drift, followed by the second set of cables at 2.5% drift. The earthquake excitation was such that the deformation demand was higher in the negative direction, resulting in the failure of the unretrofitted building, whereas the PEC retrofitted building showed stable hysteretic response, resisting higher forces while remaining within the maximum drift limit of 2.5%. The PEC system of retrofit also showed that the seismic force demands would not be increased up to 1.5% drift ratio within the boundaries of seismic response to moderate to low levels of earthquakes. Similar comparison is made for the 2-storey building. While both retrofit systems resulted in increased seismic force resistance, the PEC system showed delayed increase in force demands when needed by strong ground shaking beyond 1% lateral drift.

5.5 Use of separate footings for first-storey cables

The buildings considered in preceding sections had tension cables connected to frame corners, transferring brace forcing to the columns, increasing column axial compression during response and creating the possibility of developing net tension in the columns. The use of PEC system resulted in significant reductions in column axial forces, as well as column shear force demands. Further reductions can be obtained if the first storey cables are attached to separate footings, rather than the first storey columns (Khosravi and Saatcioglu (2021)b). The 6-storey building was further analyzed with the PEC braces anchored to the ground through their own foundations. Table 5-3 shows the comparisons of column shear and axial forces at the lowest two floors resulting from two analyses of the 6-storey building; i) with the first storey PEC braces attached to the first storey columns and ii) with the PEC braces anchored to their own footings in the ground. The latter arrangement reduced base shear on the column adjacent to the brace significantly (from 1618 kN to 227 kN) while eliminated column axial tension completely.

5.6. Conclusions

The following conclusions can be drawn from the analytical investigation reported in the preceding sections on the use of PEC system of seismic retrofitting of non-ductile reinforced concrete frame buildings in seismically active regions.

- Non-ductile frame buildings located in seismically active regions of Canada, designed

prior to 1970s, may show excessive deformation demands that exceed the current drift limits outlined in the NBC 2015 (NRCC 2015), potentially suffering from significant damage. The 6-storey mid-rise building designed for Vancouver, Canada, using the 1965 NBC (NRCC 1965) reached collapse by exceeding the crushing limit of first-storey column concrete and became unstable after 5% lateral drift under the Cascadia Earthquake Record scaled to match the design response spectrum of the NBC 2015 (NRCC 2015). The 2-storey low-rise building designed using the 1965 NBC (NRCC 1965) for the same location developed a maximum inter-storey drift of 3.69% and also showed significant damage under the same earthquake record.

- Seismically deficient reinforced concrete frame building can be retrofitted with the use of tension only braces that consist of high-strength steel cables placed diagonally, snug-tight, between the opposite corners of selected frame bays. The braces can be designed for the required increase in lateral strength and stiffness of the building. This results in the shortening of the building period, attracting higher seismic forces, accompanied by higher base shear demands. The braces may develop significant axial tension and/or compression in the attached columns, potentially resulting in net tension and foundation up-lift, or concrete crushing under excessive axial compression. Hence, when implemented, the lower-storey columns and foundations should be assessed for additional retrofit needs.
- The use of progressively engaging cables (PEC system of retrofit), where the cables remain loose until they participate at pre-selected design drift levels, showed stable hysteretic response, fulfilling the retrofit objectives without increasing seismic force demands until they are needed to control lateral drift. The cables do not engage in seismic resistance during frequent seismic events of low to intermediate intensity, and do not increase seismic force demands in this range of seismicity until the earthquake intensity is increased and additional lateral force resistance is needed. The delay in seismic force resistance results in increased building deformability with reduced base shear, as lateral force resistance is transferred from decaying frame elements to the braces. The PEC system of retrofit also develops reduced brace forces, controlling

additional axial forces applied on the adjacent columns. Both the 6-storey mid-rise and the 2-storey low-rise buildings considered for investigation developed significant seismic force resistance at maximum drift demands of 2% to 2.5% when analyzed under the NBC 2015 (NRCC 2015) compatible seismic excitation. The PEC braces remained elastic, contributing towards self centering of the building after the earthquake.

- Additional axial forces induced by tension braces on adjacent columns can be eliminated if the braces at the first-storey level are anchored to separate footings provided to support the braces. This method of anchoring the braces also reduce column base shear significantly.

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Table 5-1 Summary of dynamic analysis results for un-retrofitted buildings

EQ Records	Time History Record	Mid-rise (6-Stories)			Low-rise (2-Stories)		
		Maximum Lateral Displacement	First Storey Lateral Drift (%)	Max Base-Shear (kN)	Maximum Lateral Displacement (mm)	First Storey Lateral Drift (%)	Max Base-Shear (kN)
Cascadia	CW1	142	3.15	7170	147	3.26	2932
	CW2	98	2.17	6734	77	1.72	2725
	CW3	220 >	>5	7226	166	3.69	3029
	CW4	83	1.84	7003	81	1.81	2769
Long Records	LW1	47	1.04	5906	59	1.3	2641
	LW2	72	1.59	6518	59	1.3	2645
	LW3	63	1.4	6391	59	1.3	2692
	LW4	68	1.5	6640	56	1.25	2644
Short Records	SW1	62	1.38	6757	43	0.95	2110
	SW2	27	0.6	4240	73	1.62	2801
	SW3	26	0.57	4031	60	1.34	2572
	SW4	41	0.9	5752	50	1.11	2264

Table 5-2 Maximum response quantities in retrofitted buildings

Building Type	Tension Cable Braces	Brace Arrangement	Max. Base Shear (kN)	Max. Column C1 Axial Tension Force (kN)*	Max. Cable Stress (MPa)		Max. Storey Drift (%)	
					1 st Floor	2 nd Floor	1 st Floor	2 nd Floor
6 Storey	1860 MPa Strands	Arrangement 1	15363	2839	1052	1001	1.36	1.59
6 Storey	PEC	Arrangement 1	11749	165	730	534	2.58	2.05
6 Storey	PEC	Arrangement 2	11749	-950	730	534	2.58	2.05
2 Storey	1860 MPa Strand	Arrangement 1	5599	646	1858	770	1.91	1.06
2 Storey	PEC	Arrangement 1	4919	326	1279	186	2.3	1.19

* Positive value for axial load indicates tension.

** For unretrofitted six storey and two storey buildings, the maximum axial tensile force are -1030 and -294 Respectively.

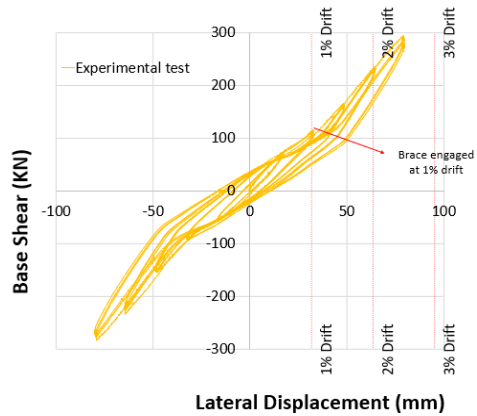
Table 5-3 Base shear and column axial forces resulting from two different PEC arrangements.

PEC Braces connected to first-storey column base				PEC Braces anchored to ground through their own footings			
Base Shear force (kN)		Max Axial Force on Columns (kN)		Base Shear force (kN)		Max Axial Force on Columns (kN)	
Total	at Column (C1)	1 st Floor (C1)	2 nd Floor (C2)	Total	at Column (C1)	1 st Floor (C1)	2 nd Floor (C2)
11749	1618	165 (T)*	-330 (C)*	11749	227	-867 (C)*	-1721 (C)*

*(T) indicates tension force ; (C) indicates compression force.



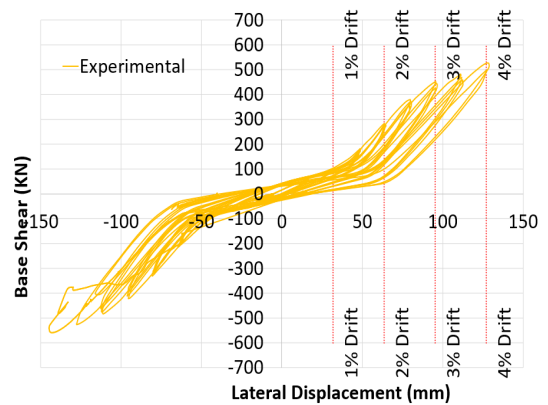
(a) Test 1 with single cables



(b) Hysteretic relationship from Test 1



(c) Test 2 with single cables



(d) Hysteretic relationship from Test 2

Figure 5-1 Frame tests and hysteretic response

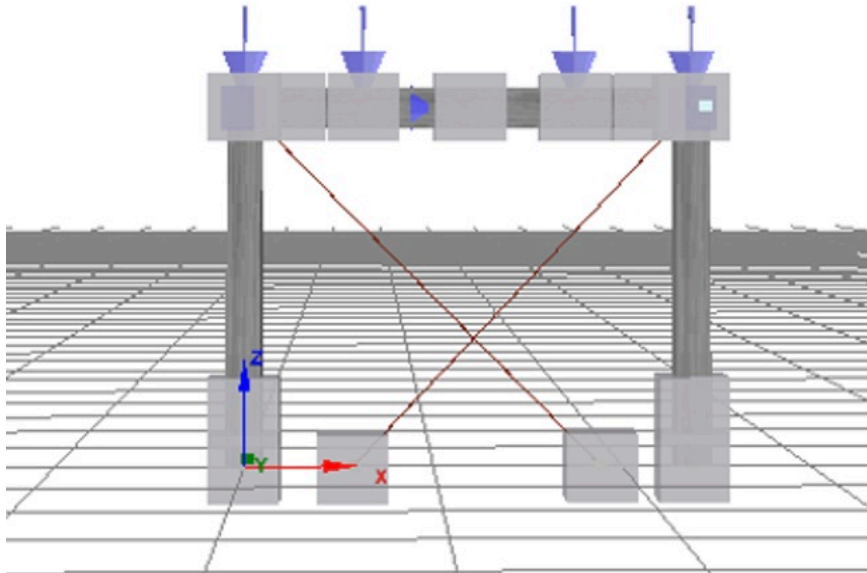
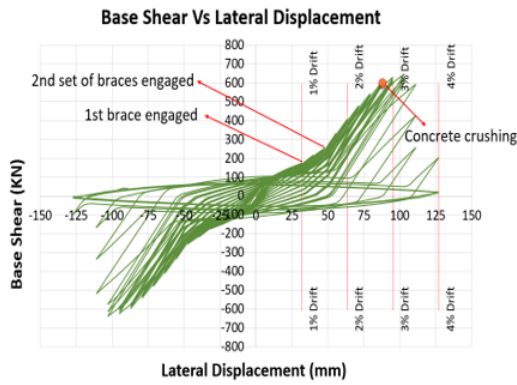
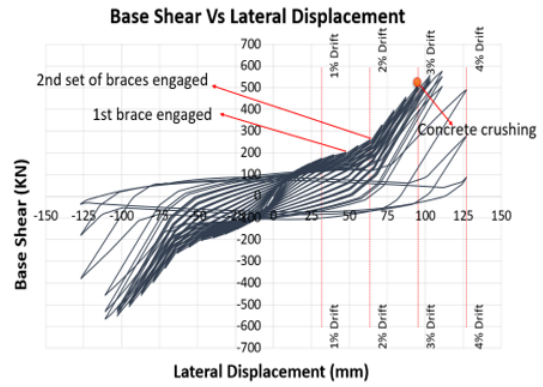


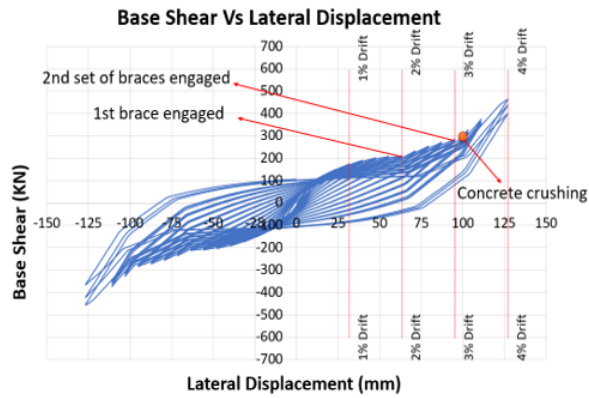
Figure 5-2 Analytical model for test frame



(a) 1st strand engaging at 1% drift; 2nd and 3rd strands engaging at 1.5% drift

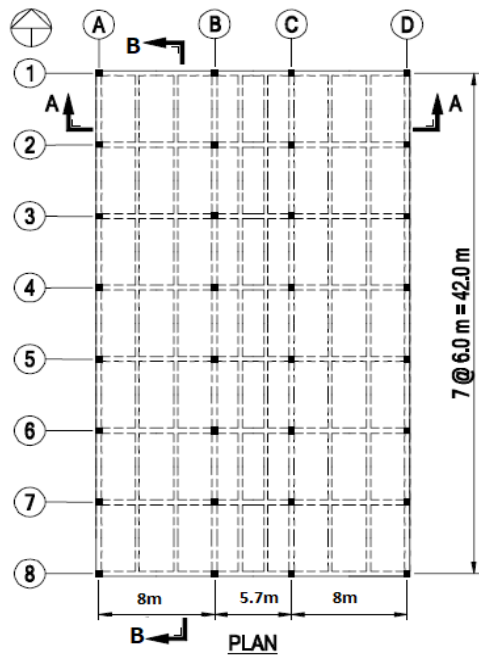


(b) 1st strand engaging at 1.5% drift; 2nd and 3rd strands engaging at 2.0% drift

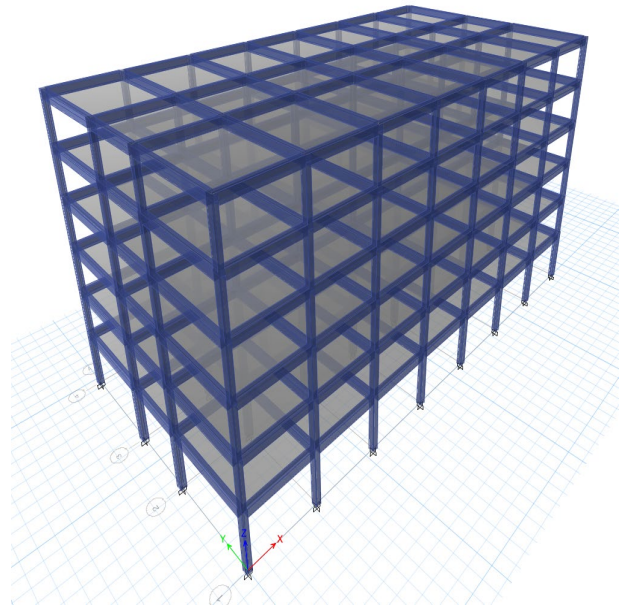


(c) 1st strand engaging at 2.0% drift; 2nd and 3rd strands engaging at 3.0% drift

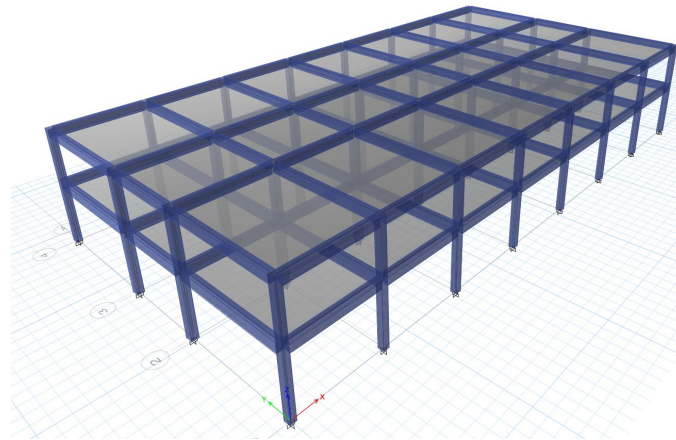
Figure 5-3 Analyses of test frame and hysteretic responses



(a) Plan view



(b) ETABS Model for 6-storey building



(c) ETABS Model for 2-storey building

Figure 5-4 Floor and elevation views of buildings selected for analysis

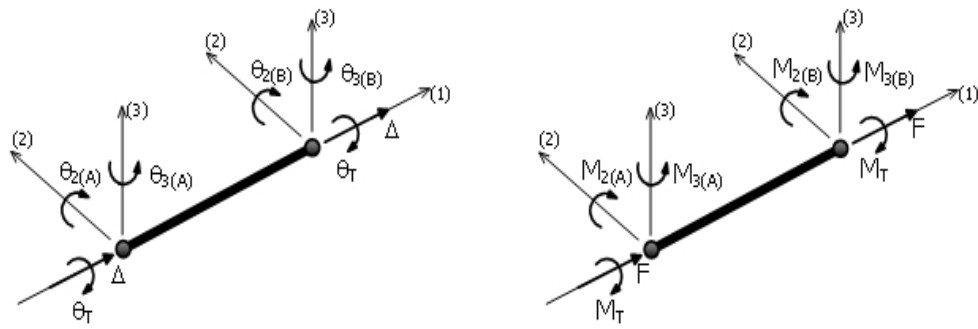
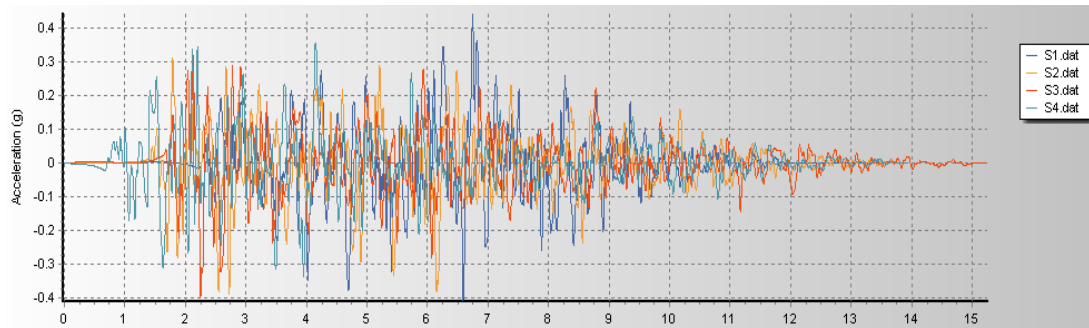
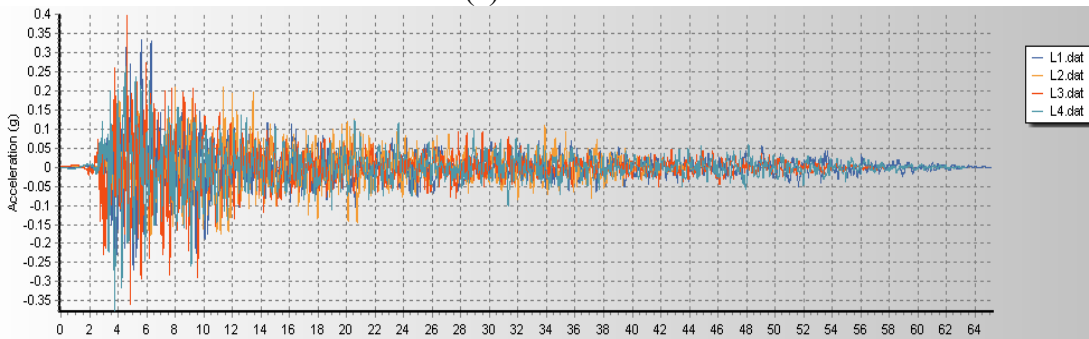


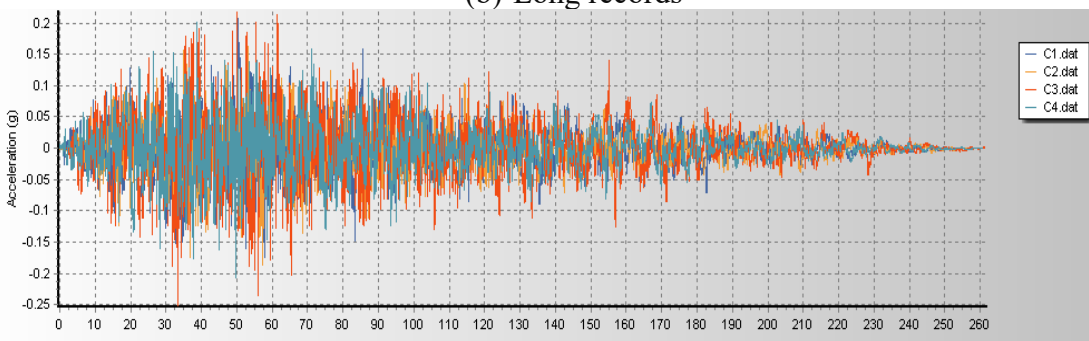
Figure 5-5 Degrees of freedom at both ends of an element



(a) Short records

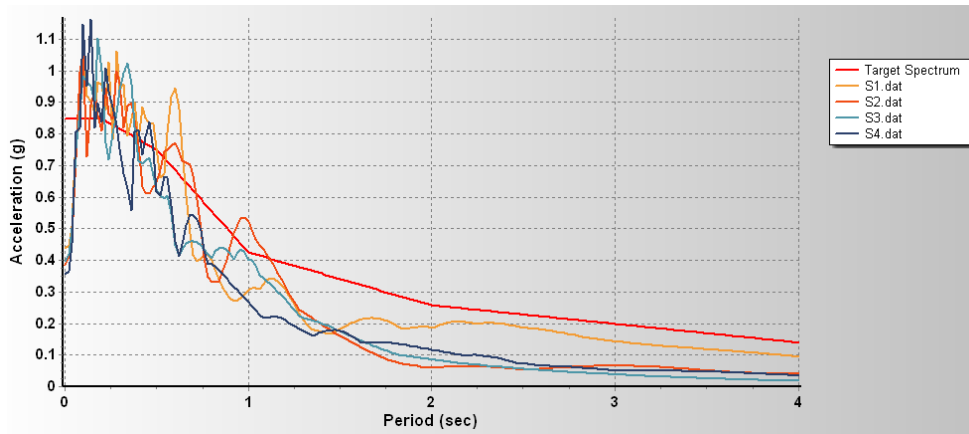


(b) Long records

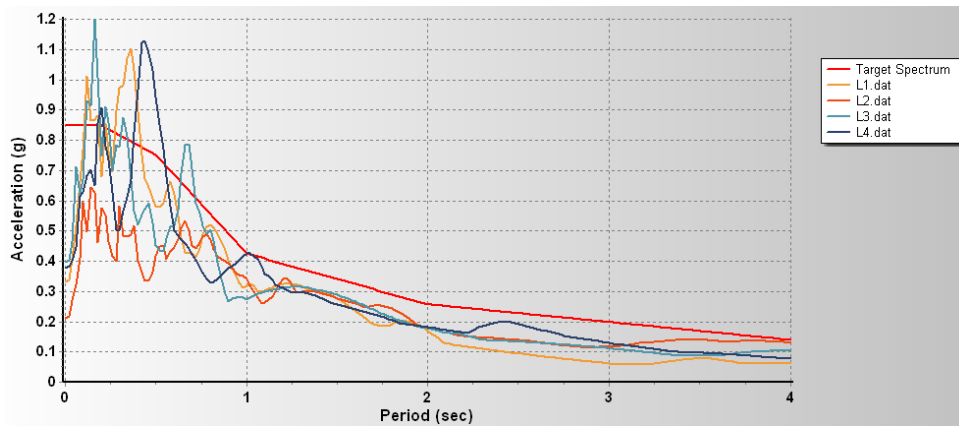


(a) Cascadia records

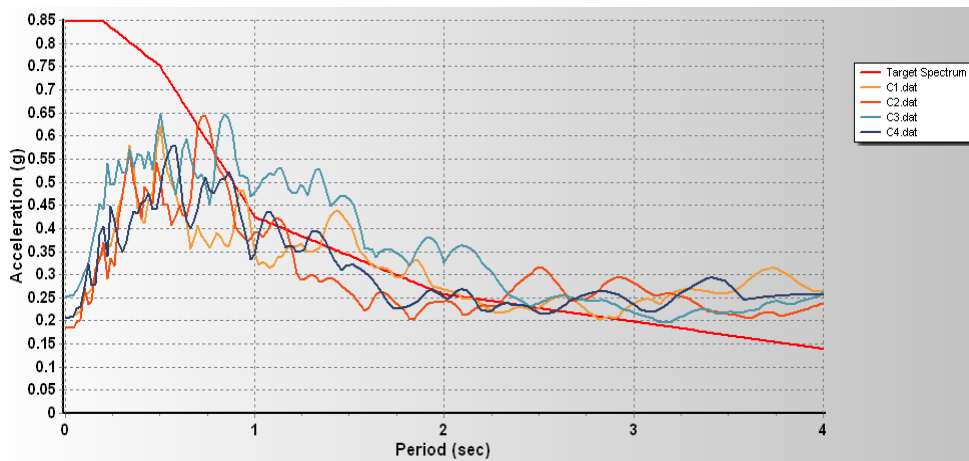
Figure 5-6 Artificial seismic time history records for Vancouver used in the analysis



(a) Short records

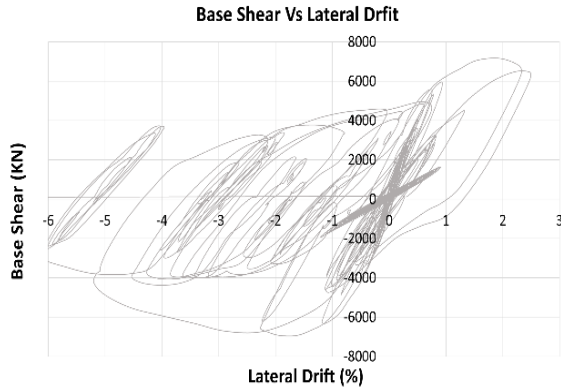


(b) Long records

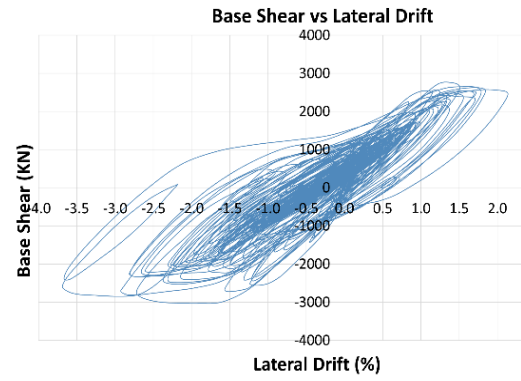


(a) Cascadia records

Figure 5-7 Comparisons of response spectra with target design spectrum of NBC 2015 for Vancouver

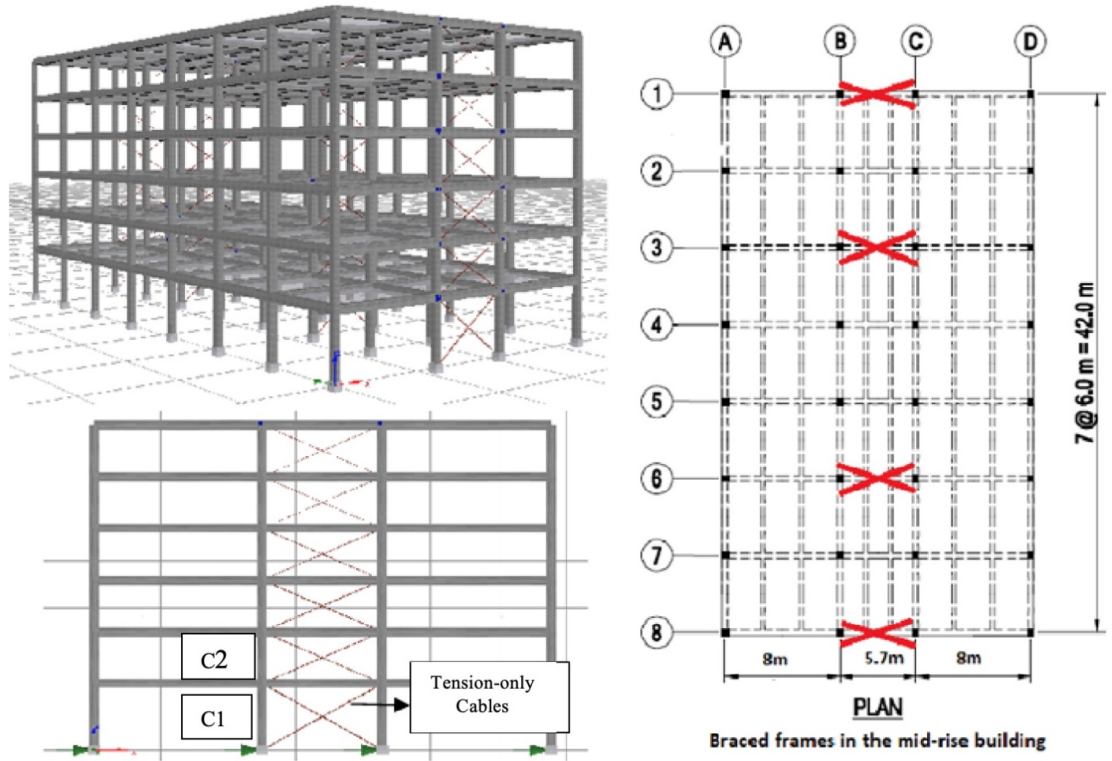


(a) 6-Storey mid-rise building

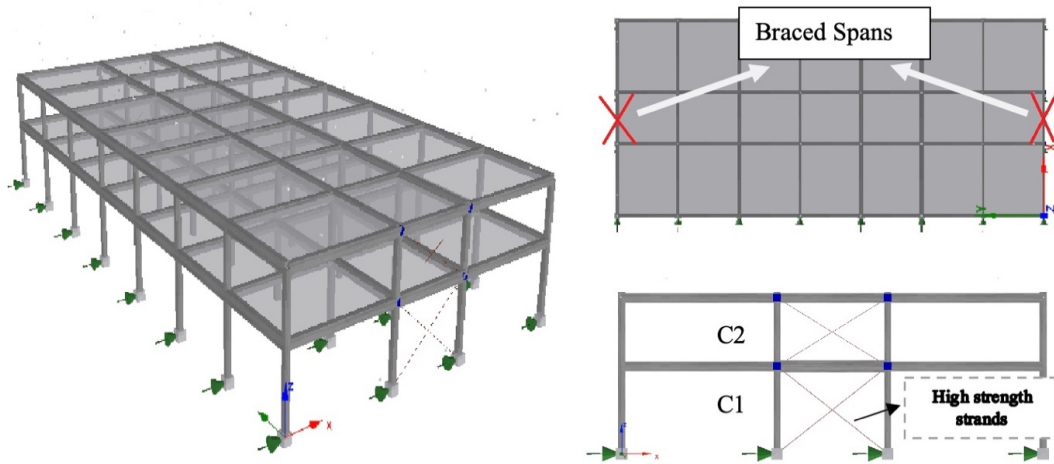


(b) 2-Storey low-rise building

Figure 5-8 Total base shear versus first-story drift hysteretic responses of unretrofitted buildings



(a) Retrofitted 6-storey building



(b) Retrofitted 2-storey building.

Figure 5-9 Buildings retrofitted with steel cables

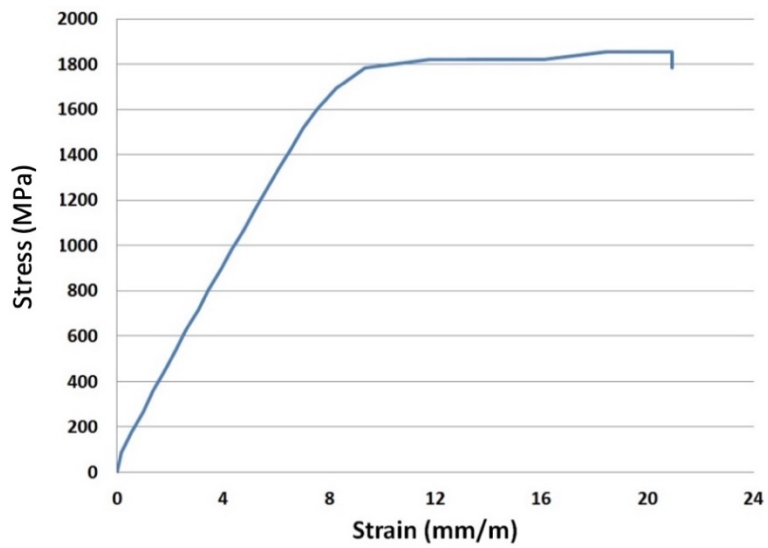
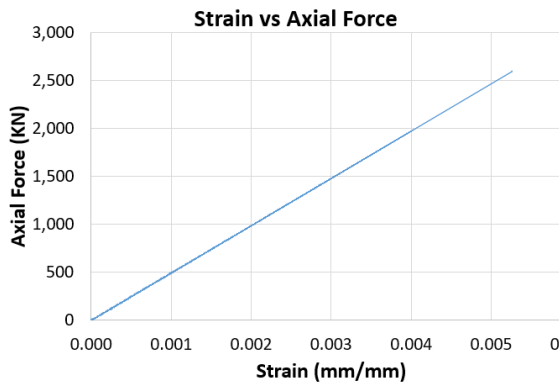
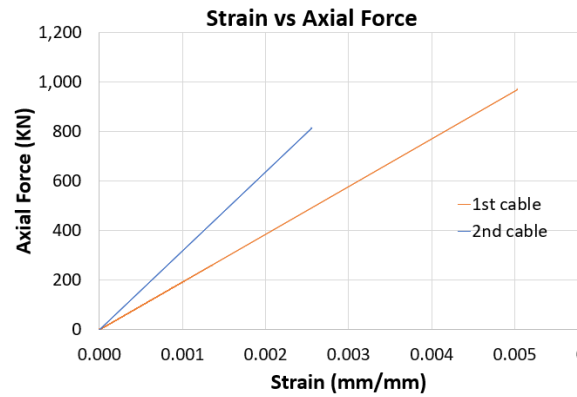


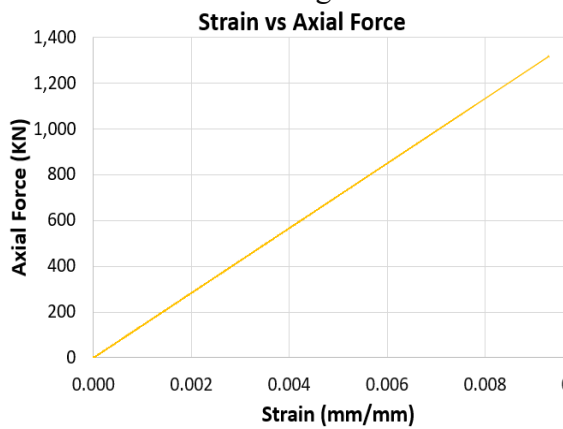
Figure 5-10 Stress-strain relationship of 7-wire strands



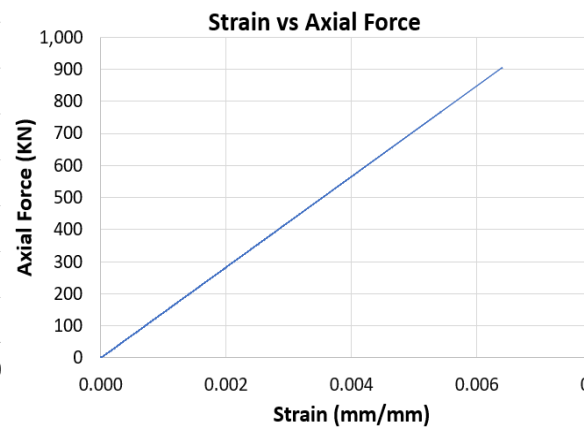
(a) Snug-tight cable brace; 6-storey building



(b) PEC system braces; 6-storey building

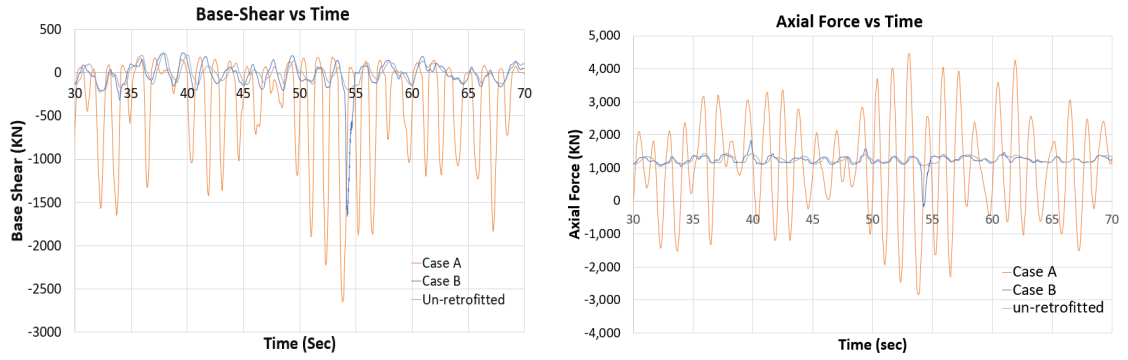


(c) Snug-tight cable brace; 2-storey building

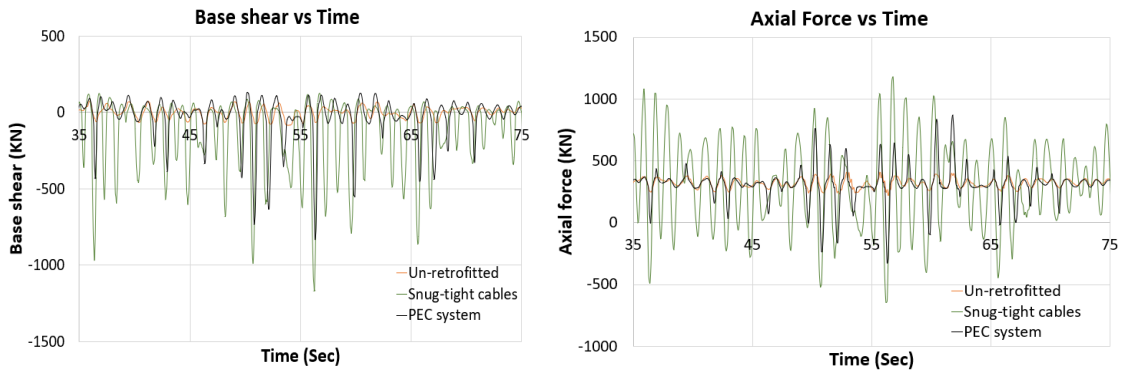


(d) PEC system brace; 2-storey building

Figure 5-11 Axial tension-axial strain relationships for tension braces at the first-floor level

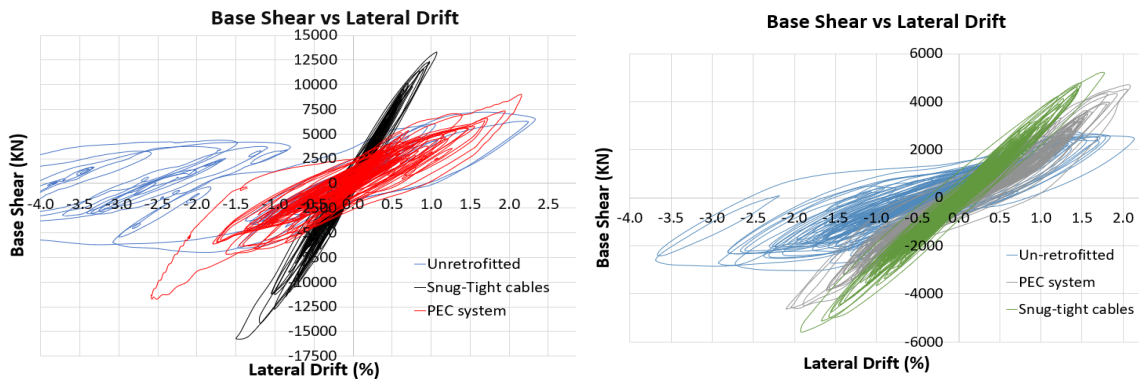


(a) 6-Storey buildings



(b) 2-Storey buildings

Figure 5-12 Time histories of shear force and axial force demands in Column C1 at the base



(a) 6-Storey building

(b) 2-Storey building

Figure 5-13 Comparisons of total base shear versus first-storey drift hysteretic relationships for the buildings analyzed

CHAPTER 6

6 Summary and Conclusion

6.1 Summary

The research project reported in this thesis has the objective of investigating the feasibility of tension-only braces for seismic retrofitting of non-ductile reinforced concrete frame buildings, while exploring the use of two innovative versions of tension braces. These include the use of shape memory alloys (SMA) with flag-shaped hysteresis loops as part of the braces, and the development and application of progressively engaging cables (PEC). The scope includes dynamic analyses of prototype buildings, selected and designed for a seismically active region in Canada to investigate the feasibility of these two retrofit strategies. It also includes an extensive experimental program to develop the PEC system of seismic retrofitting. The experimental phase of research involves two tests of a large-scale reinforced concrete frames retrofitted with different arrangement of PEC system under slowly applied lateral load reversals in the Structures Laboratory. The research findings are presented in three technical papers, included as Chapters 3, 4 and 5. The conclusions found in each phase of research are provided in the next section.

6.2 Conclusions

The following conclusions can be drawn from the analytical investigation conducted on the use of tension cable systems involving the use of high-strength 7-wire strands and mild steel cables, as well as 7-wire strand-SMA combinations as seismic retrofit systems, reported in Chapter 3.

- Pre-1971 non-ductile reinforced concrete frame buildings located in regions of high seismicity in Canada lack seismic force and deformation capacities required by modern buildings codes based on the current seismicity, as well as the current design and detailing requirements. These buildings, when subjected to current code level seismic forces, suffer from excessive inelastic storey drifts and associated damage.
- Seismically deficient older buildings can be retrofitted by means of tension cable braces provided in selected bays in two diagonal directions. Tension cables may

consist of 7-wire 1860 MPa prestressing strands, or 400 MPa mild steel cables.

- When 7-wire strand are used as tension braces, they provide effective drift control and increased lateral shear resistance. However, when attached to the corners of the frames, they may induce excessive axial tension and compression on the adjacent columns, potentially resulting in uplift and/or early crushing of column concrete. They may dissipate limited energy but develop permanent deformations.
- When Grade 400 MPa mild steel is employed as tension braces, significant yielding and energy dissipation may occur in the cable, while drift control and increased lateral force resistance may be realized. However, because Grade 400 MPa steel may experience early yielding due to its relatively low yield strength, they must be designed with care so that the non-ductile frames in the system do not experience failures as the cables yield and develop higher drift demands.
- An alternative to steel cable braces is the use of the combination of steel cables with SMA bars. The flag-shaped hysteretic response of SMA results in significant dissipation of energy, self-centering capabilities that enable the recovery of plastic deformations, minimizing pinching of hysteresis loops. This system can also generate drift control and increase seismic force resistance.
- While the use of tension cable systems is easier to construct with less interference with the existing structure, it may cause significant axial tension and compression in the attached columns. This can be minimized by using smaller number of cables in larger number of bays, or by anchoring the cables to newly created footings at the ground level, transferring cable forces directly to their foundation, rather than imposing them on the nearby columns.

The following conclusions can be drawn from the combined experimental and analytical investigation conducted for developing progressively engaging cable (PEC) system of seismic retrofitting for non-ductile reinforced concrete frame structures, reported in Chapter 4.

- The new tension-only bracing system that was developed for reinforced concrete frame

structures, consisting of progressively engaging cables (PEC), demonstrated the desired features of seismic strengthening when needed during response, lateral drift control and improved deformability. Tests of large-scale frames retrofitted with the PEC system, designed to engage in lateral force resistance at higher drift levels proved the concept of controlled participation in response thereby increasing seismic resistance when needed at higher drift levels without increasing seismic force demands during initial response when the frames alone have sufficient resistance.

- Delayed participation of the PEC system in seismic response results in increased deformability of retrofitted frames. Seismic force resistance of the PEC braces gradually increase as seismic forces are transferred from the decaying response of the non-ductile frame to the brace system, which assumes a higher role in the overall seismic force resisting mechanism of the structure.
- Tightly connected tension-only braces, unlike the PEC system, start resisting seismic forces as soon as the frame is subjected seismic excitations, stiffening the structure, and shortening its period with potentials for increasing seismic force demands. This results in increased axial tension and compression in the attached columns, potentially having detrimental effects at small lateral drift ratios in terms of net column tension, potentials for foundation lift-up and/or column concrete crushing, thereby limiting frame deformability. However, these systems may be effective for increased seismic force resistance and drift control, so long as other detrimental effects are controlled through precautionary measures.
- Multi-cable PEC systems may result in self centering of the frame through elastic response of some of the cables. The use of high-strength 7-wire strands enables the development of elastic response in some of the cable, while other may yield and dissipate energy.

The following conclusions can be drawn from the analytical investigation reported in the Chapter 5 on the use of PEC system of seismic retrofitting of non-ductile reinforced concrete frame buildings in seismically active regions.

- Non-ductile frame buildings located in seismically active regions of Canada, designed prior to 1970s, may show excessive deformation demands that exceed the current drift limits outlined in the NBC 2015 (NRCC 2015), potentially suffering from significant damage. The 6-storey mid-rise building designed for Vancouver, Canada, using the 1965 NBC (NRCC 1965) reached collapse by exceeding the crushing limit of first-storey column concrete and became unstable after 5% lateral drift under the Cascadia Earthquake Record scaled to match the design response spectrum of the NBC 2015 (NRCC 2015). The 2-storey low-rise building designed using the 1965 NBC (NRCC 1965) for the same location developed a maximum inter-storey drift of 3.69% and also showed significant damage under the same earthquake record.
- Seismically deficient reinforced concrete frame building can be retrofitted with the use of tension only braces that consist of high-strength steel cables placed diagonally, snug-tight, between the opposite corners of selected frame bays. The braces can be designed for the required increase in lateral strength and stiffness of the building. This results in the shortening of the building period, attracting higher seismic forces, accompanied by higher base shear demands. The braces may develop significant axial tension and/or compression in the attached columns, potentially resulting in net tension and foundation up-lift, or concrete crushing under excessive axial compression. Hence, when implemented, the lower-storey columns and foundations should be assessed for additional retrofit needs.
- The use of progressively engaging cables (PEC system of retrofit), where the cables remain loose until they participate at pre-selected design drift levels, showed stable hysteretic response, fulfilling the retrofit objectives without increasing seismic force demands until they are needed to control lateral drift. The cables do not engage in seismic resistance during frequent seismic events of low to intermediate intensity, and do not increase seismic force demands in this range of seismicity until the earthquake intensity is increased and additional lateral force resistance is needed. The delay in seismic force resistance results in increased building deformability with reduced base shear, as lateral force resistance is transferred from decaying frame elements to the

braces. The PEC system of retrofit also develops reduced brace forces, controlling additional axial forces applied on the adjacent columns. Both the 6-storey mid-rise and the 2-storey low-rise buildings considered for investigation developed significant seismic force resistance at maximum drift demands of 2% to 2.5% when analyzed under the NBC 2015 (NRCC 2015) compatible seismic excitation. The PEC braces remained elastic, contributing towards self centering of the building after the earthquake.

- Additional axial forces induced by tension braces on adjacent columns can be eliminated if the braces at the first-storey level are anchored to separate footings provided to support the braces. This method of anchoring the braces also reduce column base shear significantly.

6.3 Significance of research and contributions to the field of earthquake engineering

The review of previous research on seismic retrofit of non-ductile reinforced concrete frame buildings indicates that new seismic retrofit techniques with superior structural performance as well as ease and speed of construction are needed for enhanced seismic risk mitigation strategies for existing non-ductile concrete frames. The current investigation has the objective of developing improved tension-only steel cable braces. The shortcomings of the existing tension braces include increased lateral stiffness and associated increase in seismic force demands, while also resulting in reduced deformability. The first phase of research addresses the effectiveness of steel-cable-SMA assemblies with flag-shaped hysteretic behaviour, promoting improved energy dissipation and self centering capabilities. The application of the system is demonstrated in Chapter 3 through non-linear time history analyses and comparisons with the existing tension cable brace systems.

The second phase of research focuses on the development of a new tension bracing system consisting of progressively engaging cables (PEC). The system is developed through combined experimental and analytical research involving tests of large-scale reinforced concrete frames and inelastic analysis of frames under simulated seismic loading. This system has the advantage of fulfilling seismic retrofit objectives in term of increased

strength, drift control and improved inelastic deformability without increasing seismic force demands associated with lateral bracing. The system provides delayed resistance to seismic forces and engages in force resistance only when the capacity of the non-ductile frames is approached, without increasing seismic force demands prematurely. The effectiveness of the new system (PEC system of retrofitting) is demonstrated in Chapter 4. The PEC system is then applied to prototype buildings designed by following the pre-1970 era of design practice in a high seismic intensity region. The effectiveness of the system in field applications is demonstrated through dynamic inelastic response history analyses presented in Chapter 5.

In summary, the research program resulted in the development of two new tension cable systems with the feasibility of their applications to practice demonstrated through design and analysis of selected buildings.

6.4 Recommendation for future research:

The research program reported in this thesis involved two new applications, which requires further future research. The following tasks are recommended for future research.

- The newly developed PEC system of seismic retrofitting was developed with limited experimental research. Further testing of reinforced concrete frames with different arrangements of cables is recommended.
- Experimental and analytical investigations should be conducted using the PEC system as K, V, and inverted V braces. The effect of different brace patterns on strength and stiffness of reinforced concrete frames should be investigated with the focus on minimizing additional forces generated in the adjacent columns and foundations.
- Development of a design strategy for the use of the PEC system in new buildings. This involves further experimental and analytical research involving ductile concrete frames, braced with the PEC system.
- Research on generating design information on PEC design parameters for use in new buildings, including the quantification of design period, as well as design force modification factors (ductility related force modification factor R_d and overstrength

related force modification factor R_o).

- Experimental and analytical research to explore the possibilities of using other materials with different strength and elongation characteristics for the PEC system.
- Connection research for tension cables for use in both existing and new buildings for improved performance and constructability.
- Experimental research on the use of SMA materials in combination with steel tension cables, with emphasis placed on their connection to steel cables.
- Optimization of SMA materials for use in seismic retrofit applications with research involving cost-reduction methods for SMA because of the current prohibitably high cost of the material.

Appendices

Appendix I Design Loads

This Appendix presents the loads used in the design of the prototype building. The details of design loads, including the load combinations (NBCC 1965) for 6-storey buildings in Vancouver and Ottawa are given below. These loading are used to calculate the gravity load on the 2/3rd scaled test frame. The calculations of scaled loads on the frame are represented here.

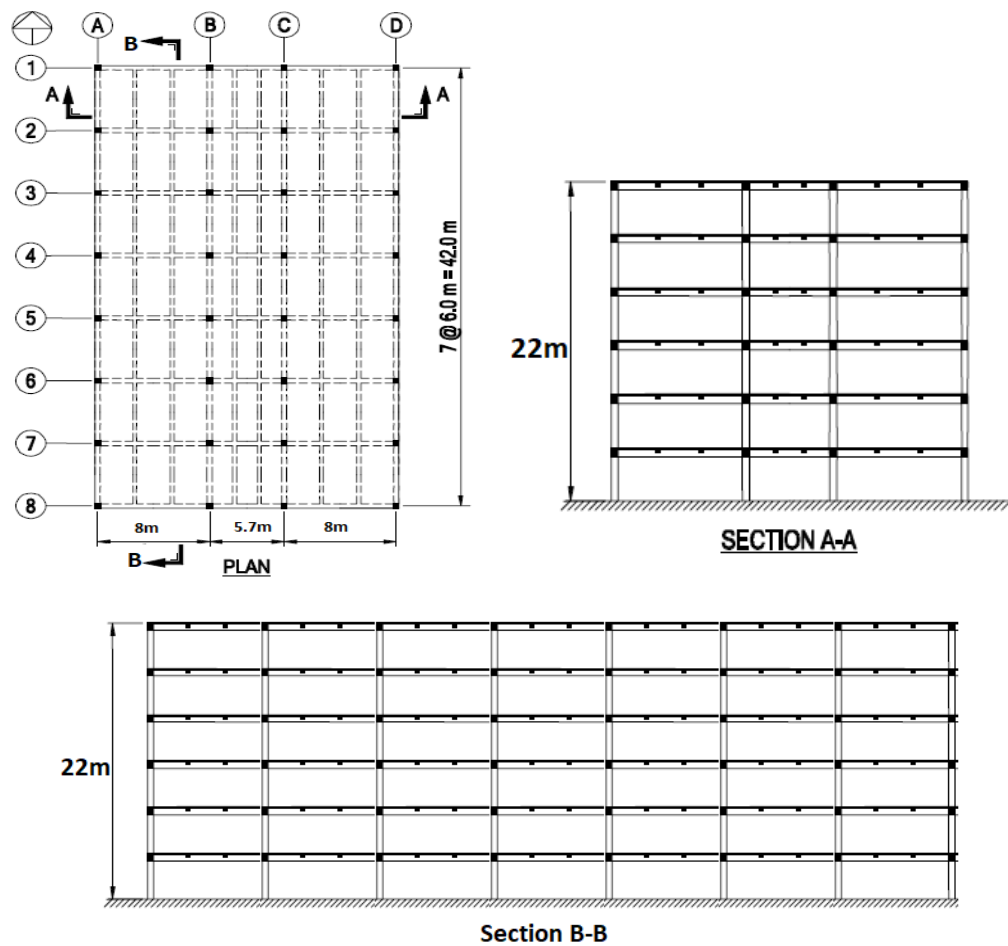


Figure I-1 Proposed prototype building plan and elevation views, located in Vancouver

Location:

Vancouver (Canada West)

Properties of Materials:

Compression strength of concrete $f'_c = 30\text{MPa}$

Yield stress of steel rods $f_y = 400\text{MPa}$

Concrete density: 24 KN/m^3

Geometry of Building:

Slab thickness: 110 mm

Interior columns: 500x500 mm

Exterior columns: 450x450 mm

Secondary beams: 300x350 mm

Main beams: 400x600 mm

Geometry of scaled Building:

1st story height: $2/3 * 4500\text{mm} = 3000\text{mm}$

Middle span length: $2/3 * 5700 = 3800\text{mm}$

Main beam dimension: $(400\text{mm} \times 600\text{mm}) \times 2/3 = 300\text{mm} \times 350\text{mm}$ (these dimensions are not exactly equal to what is calculated. The width is increased to match the column sizes)

Column dimension: $(450\text{mm} \times 450\text{mm}) \times 2/3 = 300\text{mm} \times 300\text{mm}$

Design loads and load combinations (NBCC 1965):

1) $U = 1.5 D + 1.8 L$

2) $U = 1.35 (D + L + W \text{ or } E)$

3) $U = 0.9 D + 1.35 W$

Where:

D, L, W and E represent Dead load, Live load, Wind load and Earthquake load respectively.

Summary of loads:

• Dead Loads (D):

Mechanical: 1.6 KN/m^2 distributed on central span of 5.7 m

Partitions: 1 KN/m^2

Roofing and mechanical: 1 KN/m^2

- **Live Loads (L):**

First floor: 4.9 KN/m^2

Other floors: 2.4 KN/m^2

- **Snow Loads:**

Vancouver: 1.3 KN/m^2

Note that a portion of snow load is considered as a dead load to calculate the seismic load based on NBCC 1965.

- **Wind Loads (W):**

Vancouver: 1.04 KN/m^2

Ottawa: 0.708 KN/m^2

- **Earthquake Loads (E):**

Based on NBCC code (1965)

Load calculation:

I. Beam loads

Loads Calculation in detail:

- **Dead Loads on E-W beams:**

Secondary Beams on 5.7 m span:

tributary area for a rib = $6 * (5.7/3) = 11.4 \text{ m}^2$

Mechanical and roofing: $1 * 11.4 = 11.4 \text{ KN}$

Self weight of slab with 110 mm thickness: $0.11 * 24 * 11.4 = 30 \text{ KN}$

Partitions: $1 * 11.4 = 11.4 \text{ KN}$

Beams: $0.3 * (0.35 - 0.11) * 24 * 6 = 11 \text{ KN}$

- **Live Loads on E-W beams:**

According to NBCC code and considering to the usage of stories:

$4.9 \text{ KN/m}^2 * 11.4 = 56 \text{ KN}$ (First floor)

$2.4 \text{ KN/m}^2 * 11.4 = 28 \text{ KN}$ (Other floors)

II. Columns

Tributary areas:

$$\text{Tributary area (interior 2 columns)} = (5.7+8) \times 6 = 83 \text{ m}^2$$

$$\text{Corridor tributary area} = 6 * 5.7 = 34.2 \text{ m}^2$$

Dead Loads:

$$\text{Mechanical loads on corridor: } 34.2 \times 1.6 \text{ kN/m}^2 = 54.72 \text{ kN}$$

$$\text{Mechanical and roofing: } 1 \text{ kN/m}^2 \times 83 = 83 \text{ kN}$$

$$\text{Slab self weight and partitions} = (24 \times 0.11 + 1 \text{ kN/m}^2) \times 83 = 219 \text{ kN}$$

Beam lengths and weights per unit length:

$$\text{Total beam lengths on each floor} = 23.7\text{m}$$

$$\text{Total Secondary beam lengths on each floor} = 22.4 \text{ m}$$

$$\text{Secondary beams weight per unit length} = 24 \times 0.3 \times (0.35-0.11) = 1.73 \text{ kN/m}$$

$$\text{Main beams weight per unit length} = 24 \times 0.4 \times (0.6-0.11) = 4.7 \text{ kN/m}$$

Column's weight

$$\text{Interior column self weigh per unit length} = 24 \times (0.45 \times 0.45) = 4.86 \text{ kN/m}$$

$$\text{Exterior column self weight per unit length} = 24 \times (0.5 \times 0.5) = 6 \text{ kN/m}$$

$$\text{column self weight per unit length} = (20 \times 4.86 + 12 \times 6) / 32 = 5.28 \text{ kN/m}$$

$$\text{Wight of Secondary beams: } 22.4 \times 1.73 = 39 \text{ kN}$$

$$\text{Weight of Main beams: } 23.7 \times 4.7 = 112 \text{ kN}$$

$$\text{1st floor column self weight} = 2 \text{ Columns} \times 5.28 \times (4.5- 0.11) = 46 \text{ kN}$$

$$\text{Other floors column self weight} = 2 \text{ Columns} \times 4.86 \times (3.5- 0.11) = 36 \text{ kN}$$

$$\text{Average of columns self weight for first floor} = (46+36)/2 = 41 \text{ kN}$$

Table I-1 Summary of dead load on column #2

Loads (KN) on Column #2 Grid line (7 / B to C)								
Storey	Partitions	Snow	Mechanical	Roof	Beam	Column	Slab	total load on column #2
6	0	191	55	292	151	18	219	707
5	83	0	0	219	151	36	219	560
4	83	0	0	219	151	36	219	560
3	83	0	0	219	151	36	219	560
2	83	0	0	219	151	36	219	560
1	83	0	0	219	151	36	219	560
							Sum	3582

Table I-2 Summary of dead loads on column #1

Dead load (KN) on column #1			
Storey	total load on column #1	Cumulative load	cumulative factored load
6	359	359	538.5
5	286	645	967.5
4	286	931	1396.5
3	286	1217	1825.5
2	286	1503	2254.5
1	286	1789	2683.5
Sum	1791		4478

Live Loads:

First floor live load: $83 \times 4.8 \text{ kN/m}^2 = 398 \text{ kN/m}^2$

Other Floors: $83 \times 2.4 \text{ kN/m}^2 = 199 \text{ KN}$

Here is a tabulated service live load result on the columns.

Table I-3 Live load results on the columns

Live Load (KN)	column #2 on grid line (7 / B to C)		column #1 on grid line (8 / B to C)	
	Storey	Live Load	cumulative	Live Load
6	0	0	0	0
5	199	199	100	100
4	199	398	100	200
3	199	597	100	300
2	199	796	100	400
1	199	1194	199	599

Table I-4 Cumulative load results on the columns

Cumulative load on column #1 in an interior frame				
Column on grid line 7B or 7C				
Floor	Dead Load (KN)	Live Load (KN)	2/3 rd scaled load (DL+LL) (KN)	2/3 rd Scaled factored load (1.5DL+1.8LL) (KN)
6	359	0	160	239
5	645	100	331	510
4	931	200	503	781
3	1217	300	674	1051
2	1503	400	846	1322
1	1791	597	1061	1672

The load on the exterior column will be half of the factored scaled calculated load on the interior column since the tributary area on those columns are half of interior one.

III. Earthquake Loads:

Dead Loads of Floors:

Static earthquake load can be calculated by using the formula below:

$$F_x = (V \cdot W_x \cdot h_x) / \sum_1^n (W_i h_i)$$

The weight of the floors in this formula includes 25% of snow load on the roof based on the 1965 NBCC code.

Floor area: 912 m²

Corridor area: 240 m²

Mechanical and roofing: $912 * 1 \text{ kN/m}^2 = 912 \text{ kN}$

Slab self weight: $24 * 0.11 * 912 = 2408 \text{ kN}$

Partitions: $912 * 1 \text{ kN/m}^2 = 912 \text{ kN}$

Mechanical corridor: $240 * 1.6 \text{ kN/m}^2 = 384 \text{ kN}$

Beams unit weight:

Main beams = $0.4 * (0.6-0.11) * 24 = 4.7 \text{ KN/m}$

Secondary beams = $0.3 * (0.35-0.11) * 24 = 1.73 \text{ KN/m}$

Total self weight of beams:

Secondary beams = $1.73 \text{ KN/m} * 238 \text{ m} = 412 \text{ KN}$

Main beams = $4.7 \text{ KN/m} * 313 = 1471 \text{ KN}$

Total 1883 KN

Columns unit weight:

Interior columns unit self weight = $(0.45 * 0.45) * 24 = 4.86 \text{ KN/m}$

Exterior column unit self weight = $(0.5 * 0.5) * 24 = 6 \text{ KN/m}$

Average unit weight of all columns = $(4.86 * 20 + 6 * 12) / 32 = 5.28 \text{ KN/m}$

Total self weight of columns:

1st floor = $5.28 \text{ KN/m} * (4.5- 0.11) * 32 \text{ Columns} = 742 \text{ KN}$

Other floors = $5.28 \text{ KN/m} * (3.5- 0.11) * 32 \text{ Columns} = 573 \text{ KN}$

Summation of half weight of 1st and 2nd floor columns = $(742+573)/2 = 658 \text{ KN}$

Total weight of each floor calculated is summarized here:

Table I-5 Total weight of each floor for Vancouver

Wight of stories							
Storey	Mechanical (KN) (Seible et al.)	Roofing, mechanical and partitions (KN)	Column (KN)	Beam (KN)	Slab (KN)	Snow (KN)	Weight of each storey
6	384	1824	286.5	1883	2408	Code	6785.5+0.25*Snow
5	0	1824	573	1883	2408	0	6688
4	0	1824	573	1883	2408	0	6688
3	0	1824	573	1883	2408	0	6688
2	0	1824	573	1883	2408	0	6688
1	0	1824	657	1883	2408	0	6772
						Sum	40309.5+Snow

• Earthquake Loads (E):

Base Shear (V) based on equivalent static forces NRCC (1965):

$$V = RCIFSW$$

Where;

R = Ductility factor

C = Type of Construction

I = Importance Factor

F = Soil Compressibility

S = $0.25/(9+N)$ in which N is number of stories

$$W = D + 0.25 S$$

Snow Loads for Ottawa and Vancouver:

$$1.3 \text{ KN/m}^2 * 912 = 1185.6 \text{ KN Vancouver}$$

$$W = 40606 \text{ KN Vancouver (include 25% of snow)}$$

R = 4 for Vancouver:

$$C = 1.25$$

I = 1 All other buildings not designed for post disaster

F = 1 depends on the type of the soil

$$N \text{ (Number of stories)} = 6$$

$$S = 0.25 / (9+6) = 0.017$$

V = 3265 KN Vancouver

$$F_x = (V \cdot W_x \cdot h_x) / \sum_1^n (W_i h_i)$$

Table I-6 Seismic base shear force calculations for 6 storey building in Vancouver

Floor	Height (m)	W _i (KN)	W _x .h _x (KN.m)	F _x (KN)	V (KN)
6	22	7082	155801	940	940
5	18.5	6688	123728	773.07	1688
4	15	6688	100320	626.813	2294
3	11.5	6688	76912	480.557	2785
2	8	6688	53504	334.3	3081
1	4.5	6773	30478.5	190.434	3265
	Sum	40835	540739	3265	

IV. Wind Loads:

Tributary areas of:

Interior frame tributary area of 1st floor (Axis 2-2 to 7-7): $(4.5/2 + 3.5) \times 6 = 24 \text{ m}^2$

Other floors (Axis 2-2 to 7-7): $6 \times 3.5 = 21 \text{ m}^2$

Roof (2-2 to 7-7): $6 \times 3.5/2 = 10.5 \text{ m}^2$

Table I-7 Wind load calculations

Wind Loads						
Floor	1	2	3	4	5	6
Height from ground level (m)	4.5	8	11.5	15	18.5	22
Height Factor	1	1	1	1	1	1
Wind Pressure (KN/m ²)	1	1	1	1	1	1
Tributary Area (m ²)	10.5	21	21	21	21	24
Wind Force (KN)	18	15	15	17	17	9
Cumulative Shear Force (KN)	91	73	58	43	26	9
Moment about ground level (KN.m)	81	120	173	255	314.5	198
Cumulative moment about ground level (KN.m)	1141	1060	940	768	512.5	198
cumulative shear per column (KN)	22.8	18.3	14.5	10.8	6.5	2.3
2/3rd Scaled cumulative shear (KN)	10.1	8.1	6.4	4.8	2.9	1.0

A) 2nd Storey Interior Frame 7 (B to C) Columns

Summary of Loadings:

$DL + LL = 668 + 177 = 845 \text{ kN} > 800 \text{ kN}$ (Applied load on the lab frame column) OK.

(Considering no live load reduction applied in the analysis)

B) 1st Storey Exterior Frame 8 (B to C) Columns

Summary of Loadings:

$DL + LL = 398 + 133 = 531 \text{ kN} < 800 \text{ kN}$ (Applied load on the lab frame column) OK.

Appendix II Seismic design loads, artificial seismic records and dynamic analysis result of the buildings in Ottawa

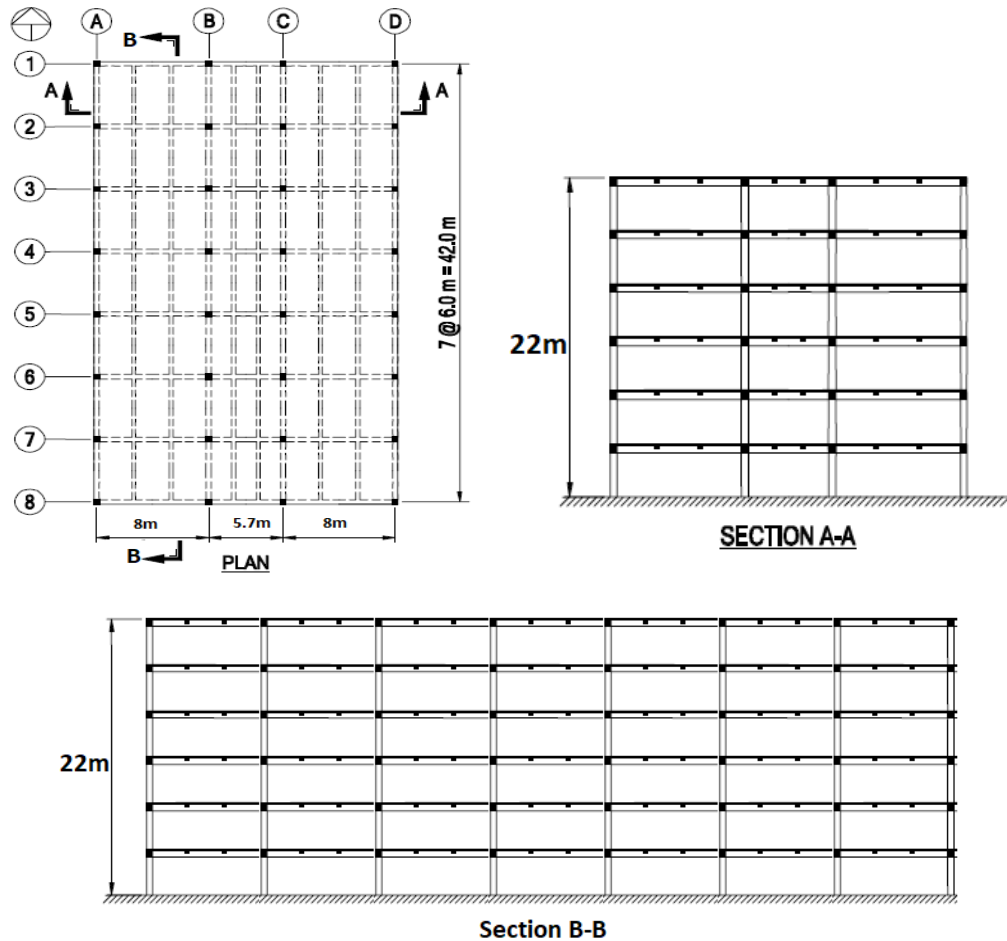


Figure II-1 Proposed prototype building plan and elevation views, located in Ottawa

Load calculation:

Beam loads

Loads Calculation in detail:

- Dead Loads on E-W beams:

Secondary Beams on 5.7 m span:

$$\text{tributary area for a rib} = 6 * (5.7/3) = 11.4 \text{ m}^2$$

$$\text{Mechanical and roofing: } 1 * 11.4 = 11.4 \text{ KN}$$

$$\text{Self weight of slab with 110 mm thickness: } 0.11 * 24 * 11.4 = 30 \text{ KN}$$

Partitions: $1 * 11.4 = 11.4$ KN

Beams: $0.3 * (0.35 - 0.11) * 24 * 6 = 11$ KN

• Live Loads on E-W beams:

According to NBCC code and considering to the usage of stories:

$4.9 \text{ KN/m}^2 * 11.4 = 56$ KN (First floor)

$2.4 \text{ KN/m}^2 * 11.4 = 28$ KN (Other floors)

Tributary areas:

Tributary area (interior 2 columns) = $(5.7+8) * 6 = 83 \text{ m}^2$

Corridor tributary area = $6 * 5.7 = 34.2 \text{ m}^2$

Dead Loads:

Mechanical loads on corridor: $34.2 * 1.6 \text{ kN/m}^2 = 54.72$ kN

Mechanical and roofing: $1 \text{ kN/m}^2 * 83 = 83$ kN

Slab self weight and partitions = $(24 * 0.11 + 1 \text{ kN/m}^2) * 83 = 219$ kN

Beam lengths and weights per unit length:

Total beam lengths on each floor = 23.7m

Total Secondary beam lengths on each floor = 22.4 m

Secondary beams weight per unit length = $24 * 0.3 * (0.35-0.11) = 1.73$ kN/m

Main beams weight per unit length = $24 * 0.4 * (0.6-0.11) = 4.7$ kN/m

Column's weight

Interior column self weigh per unit length = $24 * (0.45 * 0.45) = 4.86$ kN/m

Exterior column self weight per unit length = $24 * (0.5 * 0.5) = 6$ kN/m

column self weight per unit length = $(20 * 4.86 + 12 * 6) / 32 = 5.28$ kN/m

Wight of Secondary beams: $22.4 * 1.73 = 39$ kN

Weight of Main beams: $23.7 * 4.7 = 112$ kN

1st floor column self weight = $2 \text{ Columns} * 5.28 * (4.5 - 0.11) = 46$ kN

Other floors column self weight = $2 \text{ Columns} * 4.86 * (3.5 - 0.11) = 36$ kN

Average of columns self weight for first floor = $(46+36)/2 = 41$ kN

Live Loads:

First floor live load: $83 \times 4.8 \text{ kN/m}^2 = 398 \text{ kN/m}^2$

Other Floors: $83 \times 2.4 \text{ kN/m}^2 = 199 \text{ KN}$

Earthquake Loads:

Dead Loads of Floors:

Static earthquake load can be calculated by using the formula below:

$$F_x = (V \cdot W_x \cdot h_x) / \sum_1^n (W_i h_i)$$

The weight of the floors in this formula includes 25% of snow load on the roof based on the 1965 NBCC code.

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Mechanical corridor: $240 \times 1.6 \text{ kN/m}^2 = 384 \text{ kN}$

Beams unit weight:

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Total self weight of beams:

Secondary beams = $1.73 \text{ KN/m} \times 238 \text{ m} = 412 \text{ KN}$

Main beams = $4.7 \text{ KN/m} \times 313 = 1471 \text{ KN}$

Total 1883 KN

Columns unit weight:

Interior columns unit self weight = $(0.45 \times 0.45) \times 24 = 4.86 \text{ KN/m}$

Exterior column unit self weight = $(0.5 \times 0.5) \times 24 = 6 \text{ KN/m}$

Average unit weight of all columns = $(4.86 * 20 + 6 * 12) / 32 = 5.28 \text{ KN/m}$

Total self weight of columns:

1st floor = $5.28 \text{ KN/m} * (4.5 - 0.11) * 32 \text{ Columns} = 742 \text{ KN}$

Other floors = $5.28 \text{ KN/m} * (3.5 - 0.11) * 32 \text{ Columns} = 573 \text{ KN}$

Summation of half weight of 1st and 2nd floor columns = $(742+573)/2 = 658 \text{ KN}$

Total weight of each floor calculated is summarized here:

Table II-1 Total weight of each floor for Ottawa

Wight of stories							
Storey	Mechanical (KN) (Seible et al.)	Roofing, mechanical and partitions (KN)	Column (KN)	Beam (KN)	Slab (KN)	Snow (KN)	Weight of each storey
6	384	1824	286.5	1883	2408	Code	$6785.5 + 0.25 * \text{Snow}$
5	0	1824	573	1883	2408	0	6688
4	0	1824	573	1883	2408	0	6688
3	0	1824	573	1883	2408	0	6688
2	0	1824	573	1883	2408	0	6688
1	0	1824	657	1883	2408	0	6772
						Sum	$40309.5 + \text{Snow}$

• Earthquake Loads (E):

Base Shear (V) based on equivalent static forces NRCC (1965):

$$V = RCIFSW$$

Where;

R = Ductility factor

C = Type of Construction

I = Importance Factor

F = Soil Compressibility

S = $0.25 / (9 + N)$ in which N is number of stories

$$W = D + 0.25 S$$

Snow Loads for Ottawa:

$$2.3 \text{ KN/m}^2 * 912 = 2097.6 \text{ KN}$$

$$W = 40835 \text{ KN (include 25% of snow)}$$

R = 4 for Ottawa

C = 1.25

I = 1 All other buildings not designed for post disaster

F= 1 depends on the type of the soil

N (Number of stories) = 6

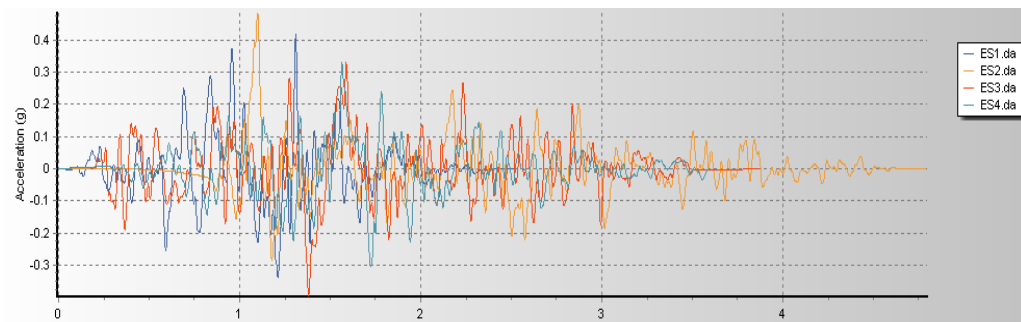
S= $0.25 / (9+6) = 0.017$

V = 3410 KN for Ottawa

Table II-2 Seismic base shear force calculations for 6 storey building in Ottawa

Floor	Height (m)	W_i (KN)	$W_x.h_x$ (KN.m)	F_x (KN)	V (KN)
6	22	7310	160820	1004.83	1004.83
5	18.5	6688	123728	773.07	1777.9
4	15	6688	100320	626.813	2404.71
3	11.5	6688	76912	480.557	2885.27
2	8	6688	53504	334.3	3219.57
1	4.5	6773	30478.5	190.434	3410
	Sum	40835	545762.5	3410	

- Artificial dynamic time history records and response spectra for Ottawa:

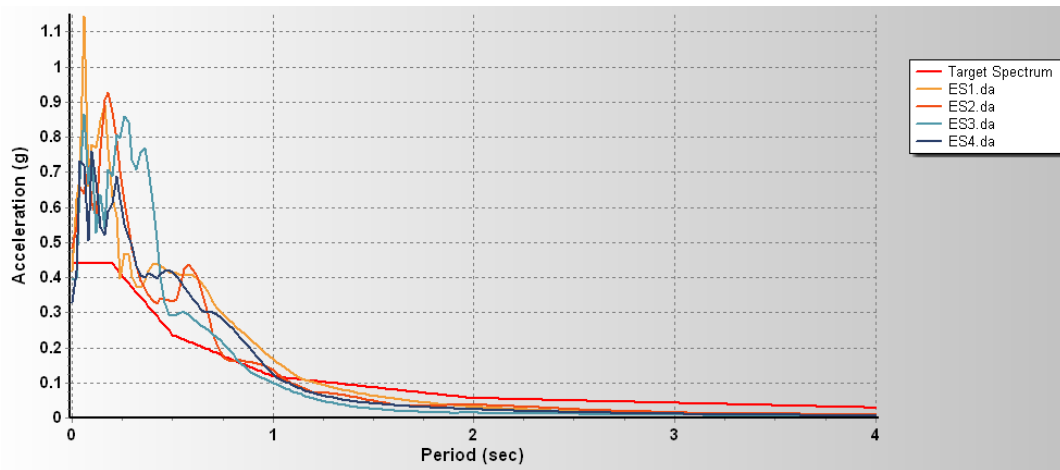


a) Short records

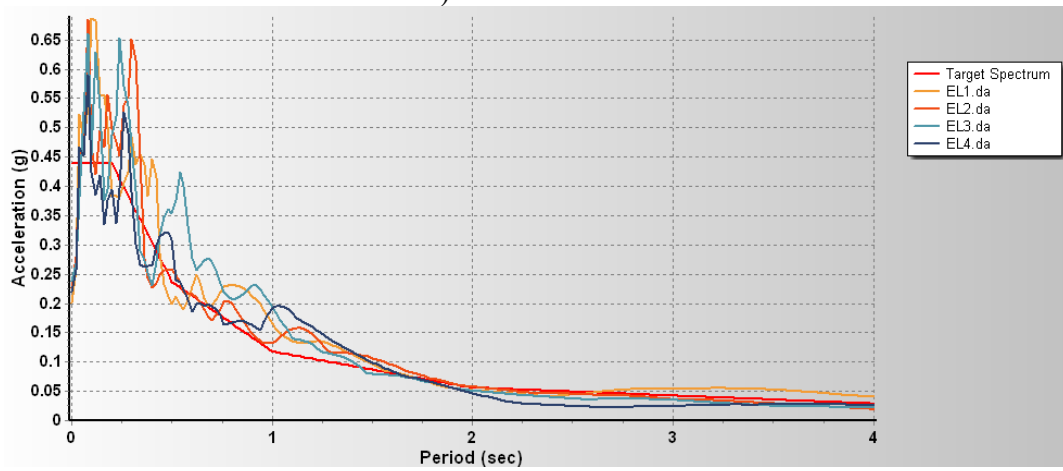


b) Long records

Figure II-2 Artificial seismic time history records for Ottawa used in the analysis



a) Short records



b) Long records

Figure II-3 Comparisons of response spectra with target design spectrum of NBC 2015 for Ottawa

Table II-3 Dynamic Analysis results for mid-rise and low-rise building in Ottawa

		Unretrofitted buildings located in Ottawa					
		Mid-rise			Low-rise		
	Time History Record	Maximum Lateral Displacement (mm)	First Storey Max Lateral drift (%)	Max base-shear (KN)	Maximum Lateral Displacement (mm)	First Storey Lateral drift (%)	Max base-shear (KN)
Long Records	LE1	17	0.38	3617	30	0.65	1797
	LE2	16	0.36	3342	22	0.50	1674
	LE3	13	0.29	4094	28	0.62	2045
	LE4	19.8	0.44	3914	19	0.42	1690
Short Records	SE1	14	0.31	2997	38	0.84	2079
	SE2	9.5	0.21	2702	22	0.48	1658
	SE3	8.7	0.21	2110	20	0.44	1655
	SE4	7	0.20	1846	28	0.62	2033

Appendix III Damage observations in Test 1 and 2

In this appendix additional observations of progressive damages in the frames during the experimental tests 1 and 2, are represented.

- **Test 1 observations:**



(a) East column

(b) West column

(c) Beam-column joint

Figure III-1 Cracks at the end of 1% drift cycles



Figure III-2 Crack pattern during Test 1 after 2% drift cycles



Figure III-3 Crack patterns at the end of 2.5% drift cycles

- **Observations in test 2:**



Figure III-4 crack pattern at the end of 2% drift cycles



Figure III-5 Beginning of cover concrete crushing and crack pattern at 2.5% drift during Test 2



Figure III-6 Increased cover concrete crushing and crack pattern at 3.0 % drift during Test 2



Figure III-7 Increased cover concrete crushing and crack pattern at 3.5 % drift during

Test 2

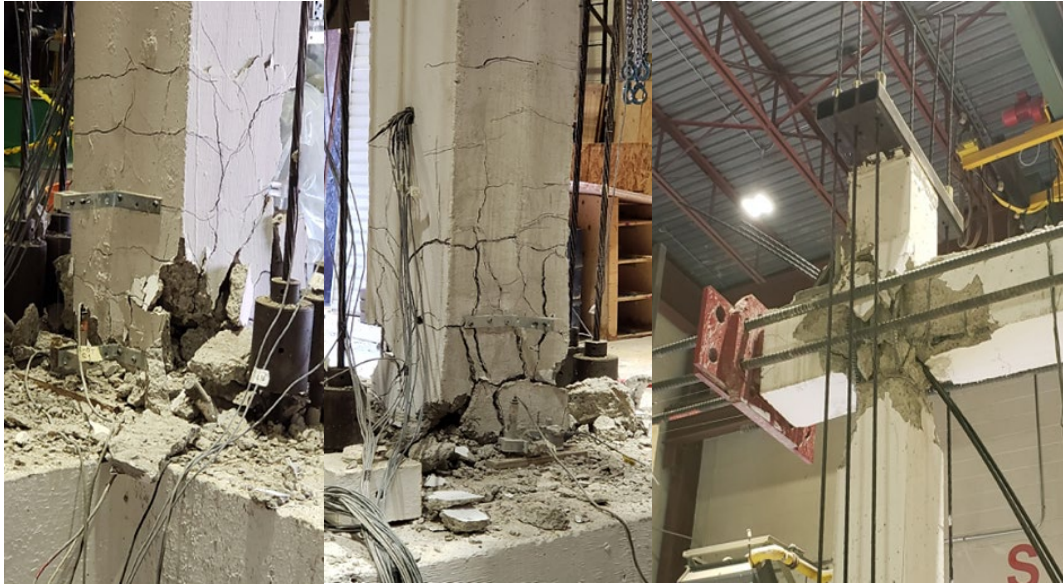


Figure III-8 Crushing of concrete and buckling of column reinforcement at 4.0 % drift during Test 2

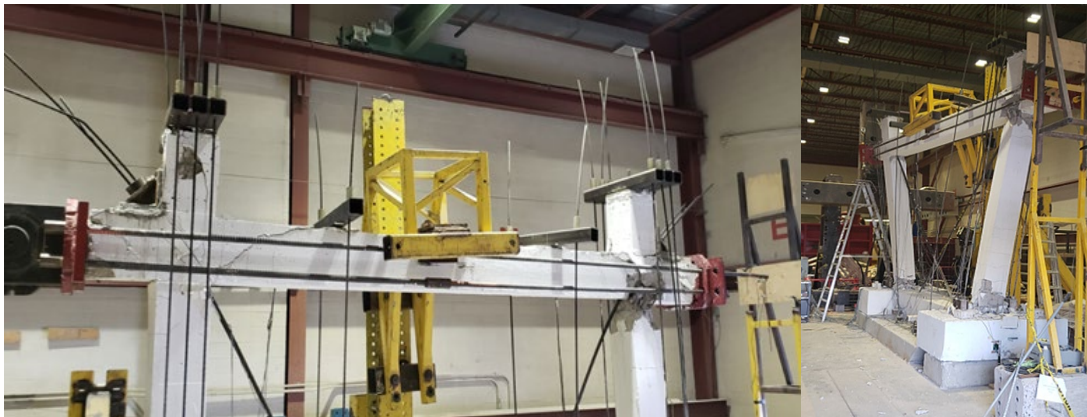
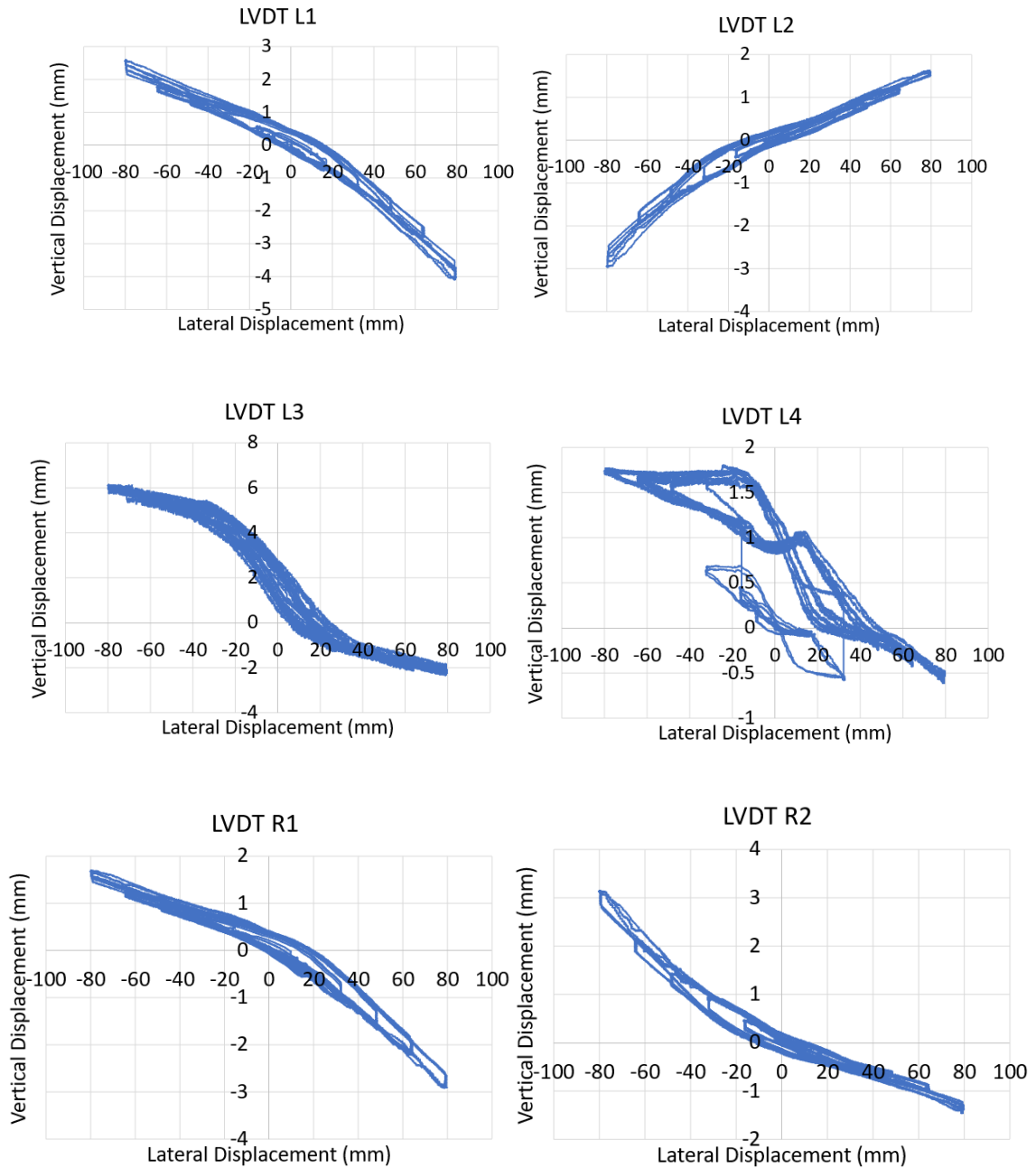


Figure III-9 Frame at the end of Test 2

Appendix IV Instrumentation Data

The data recorded from instrumentation for test 1 and test 2, is provided here:

- **Instrumentation Data of Test 1**



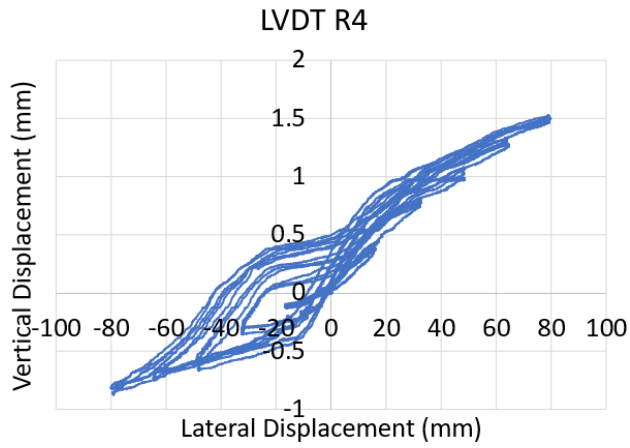


Figure IV-1 Frame lateral displacement vs. vertical displacement recorded by LVDTs in Test 1

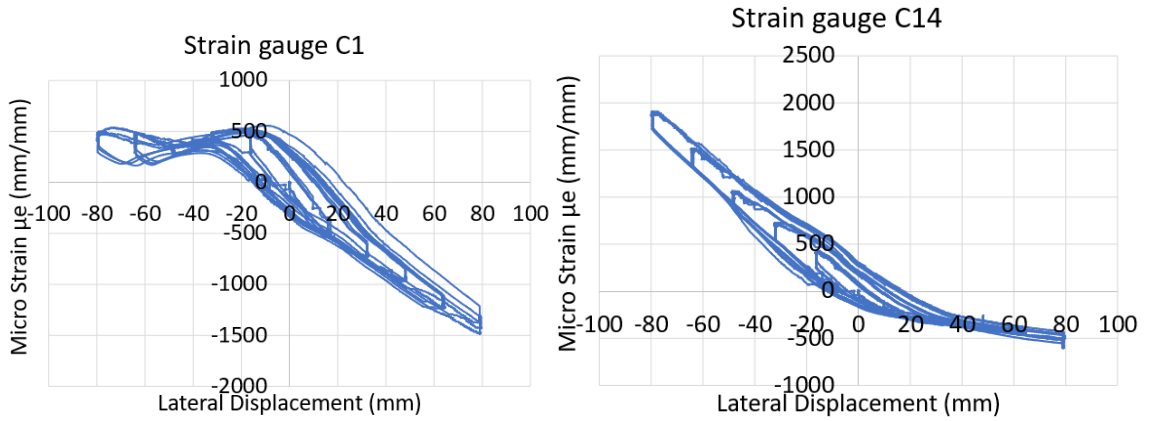
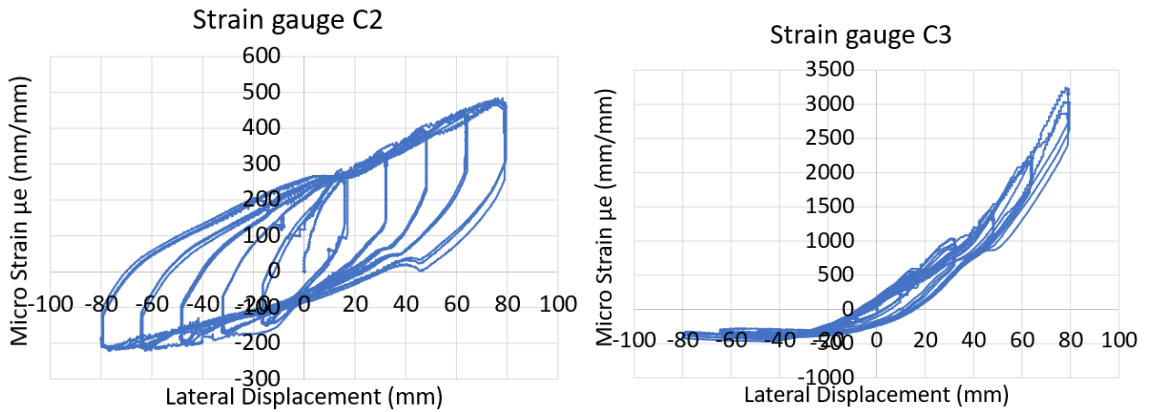


Figure IV-2 Frame lateral displacement vs. vertical displacement recorded by LVDTs in Test 1



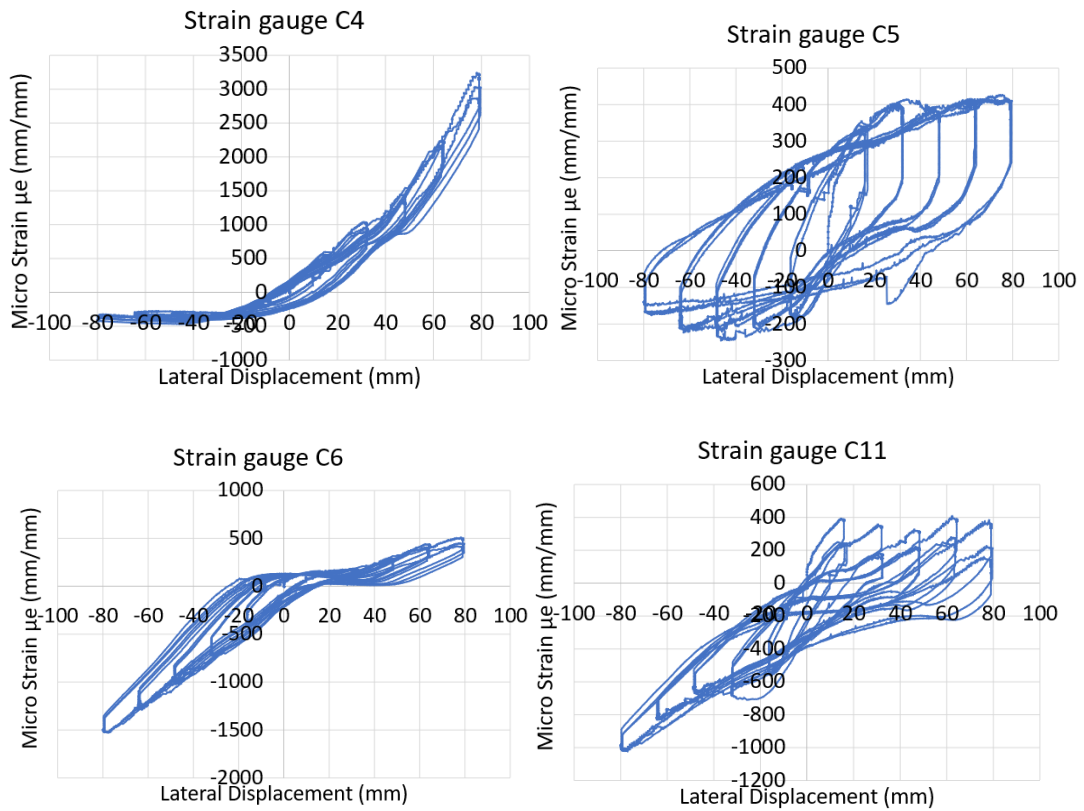
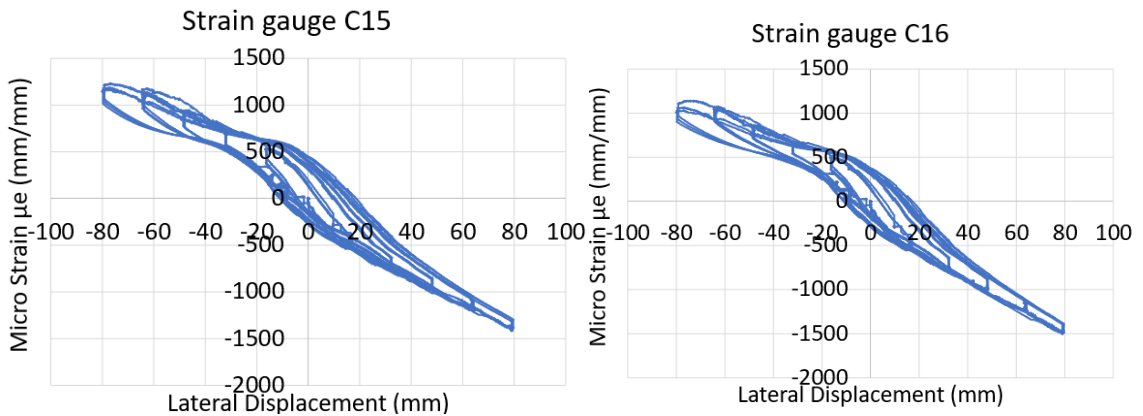


Figure IV-3 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at outer face of base of the East column.



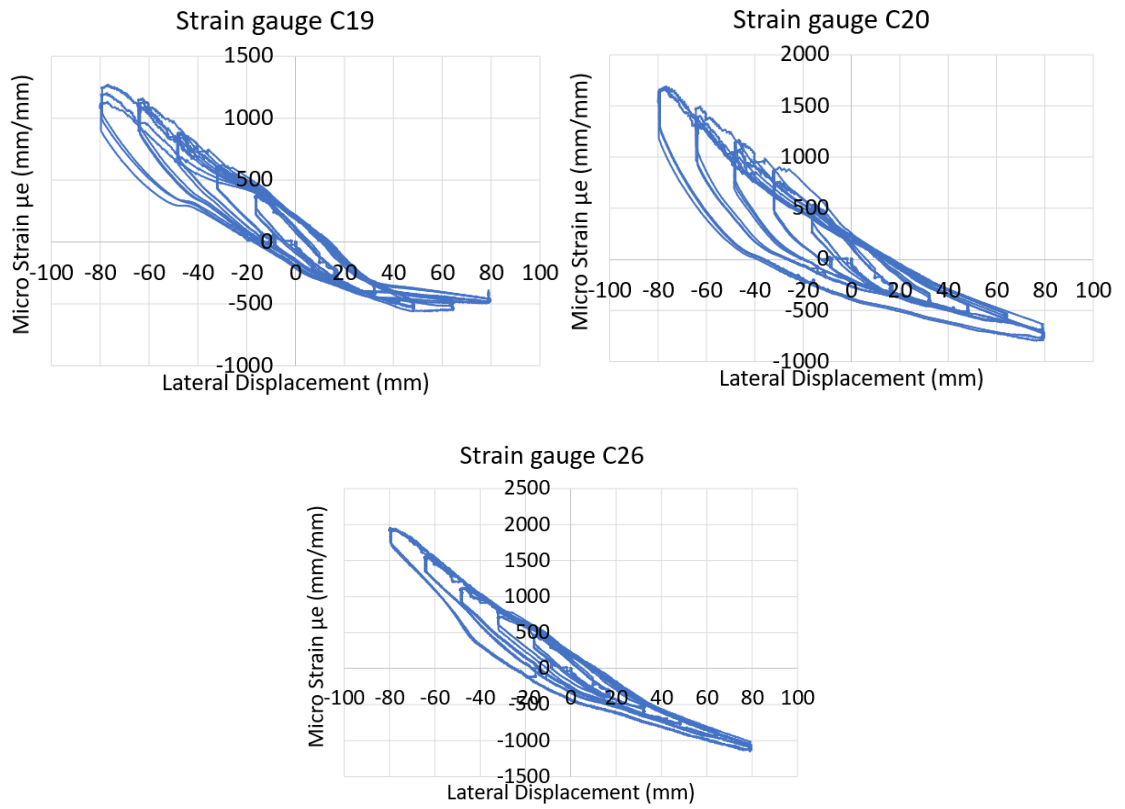
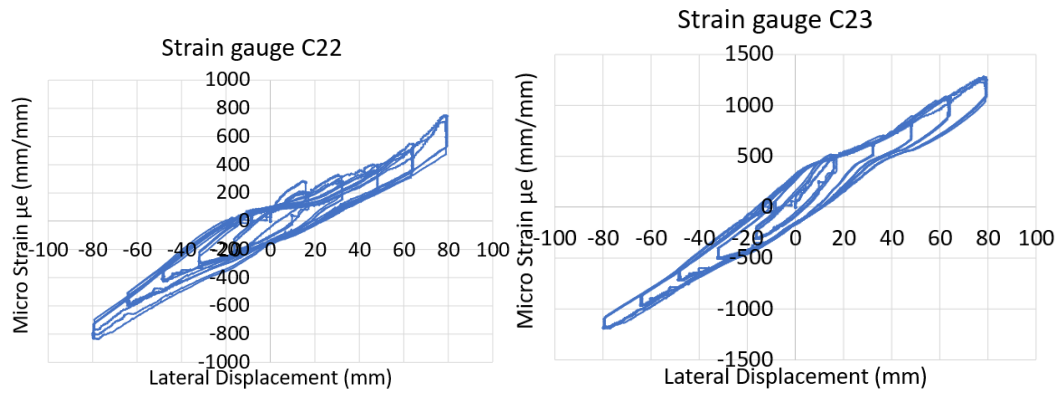


Figure IV-4 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at outer face of base of the West column.



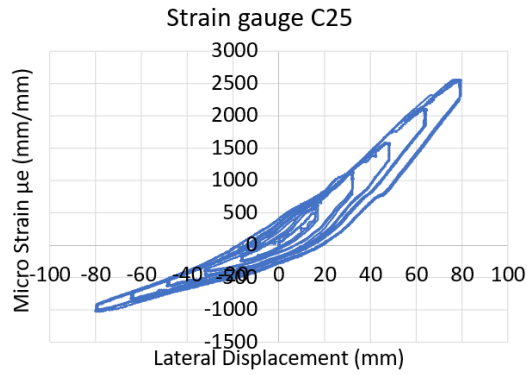


Figure IV-5 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at inner face of base of the right column.

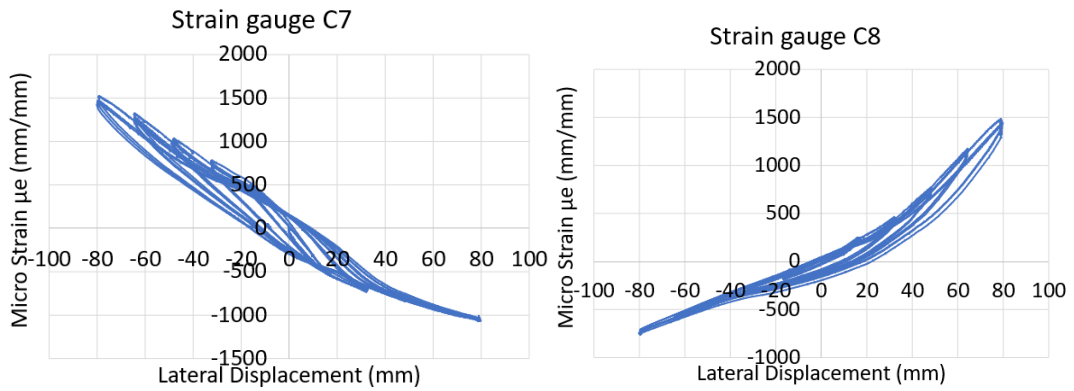


Figure IV-6 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at outer and inner face of the top of the left column

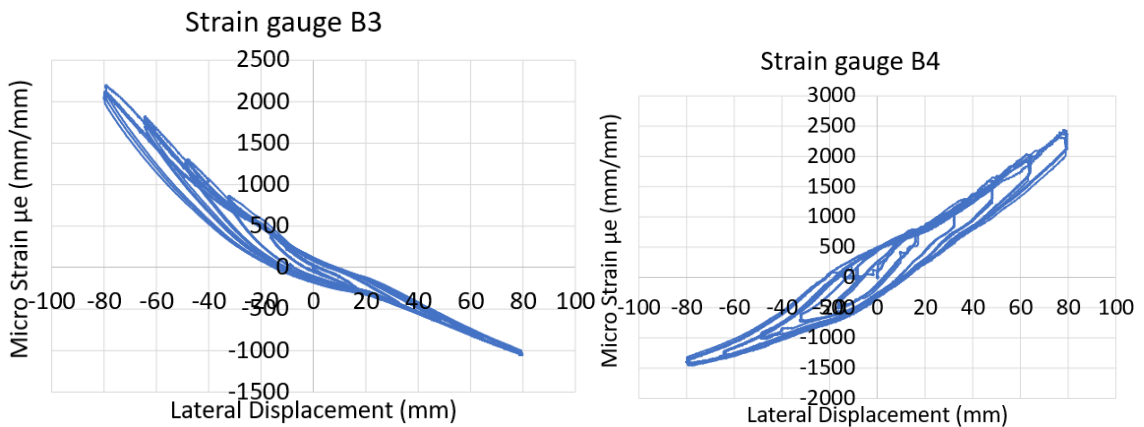


Figure IV-7 Frame lateral displacement vs. strains recorded by strain gauges on the top and bottom reinforcing bars at left side of the beam

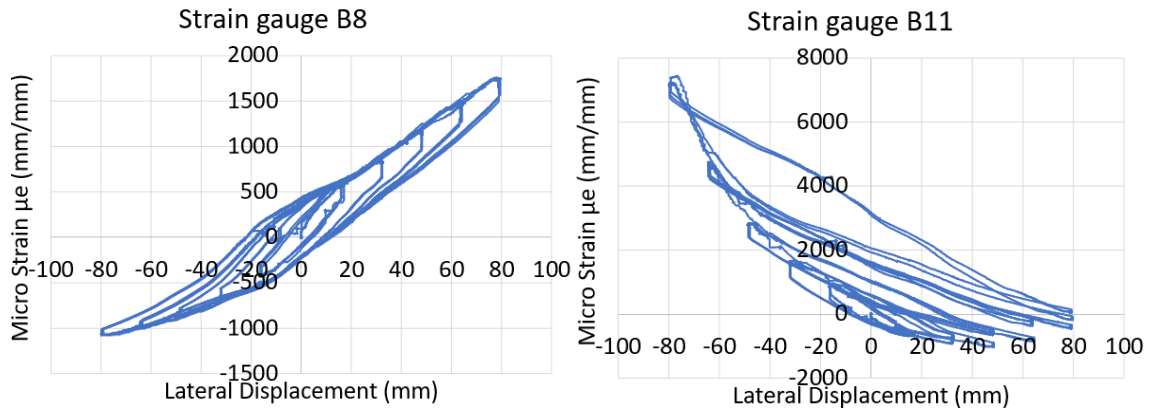
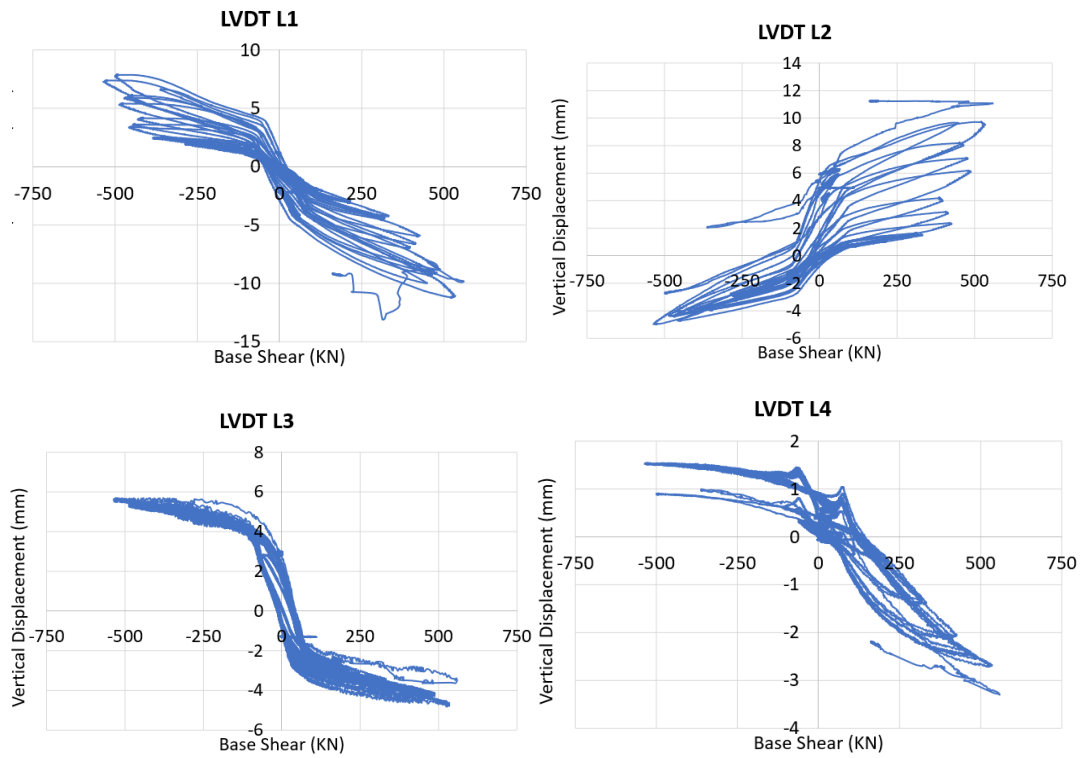


Figure IV-8 Frame lateral displacement vs. strains recorded by strain gauges on the top and bottom reinforcing bars at right side of the beam.

- Instrumentation Data of Test 2



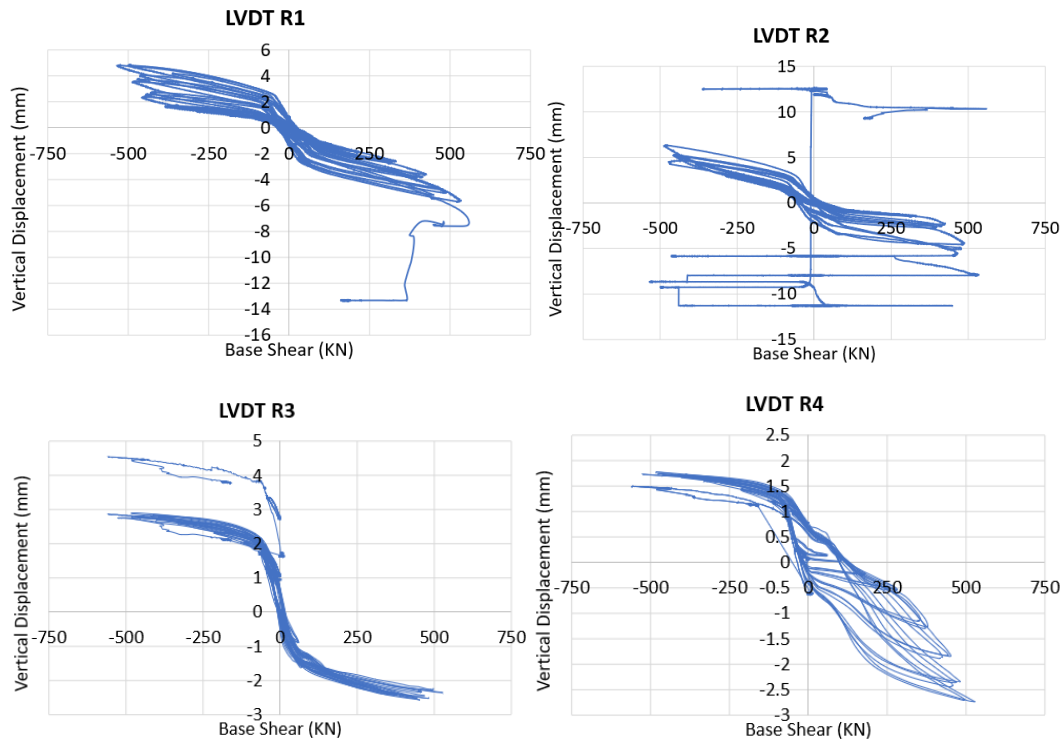


Figure IV-9 Frame lateral displacement vs. vertical displacement recorded by LVDTs in test 2

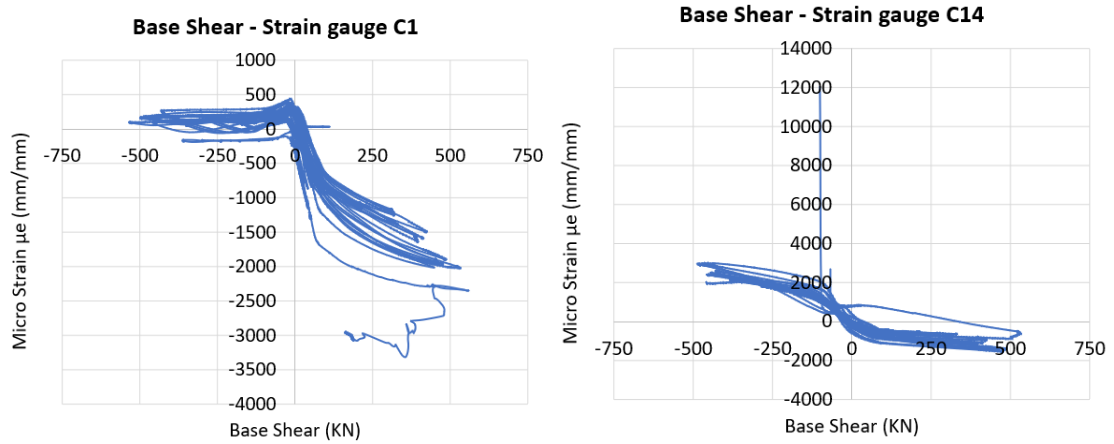


Figure IV-10 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at inner face of base of the left column.

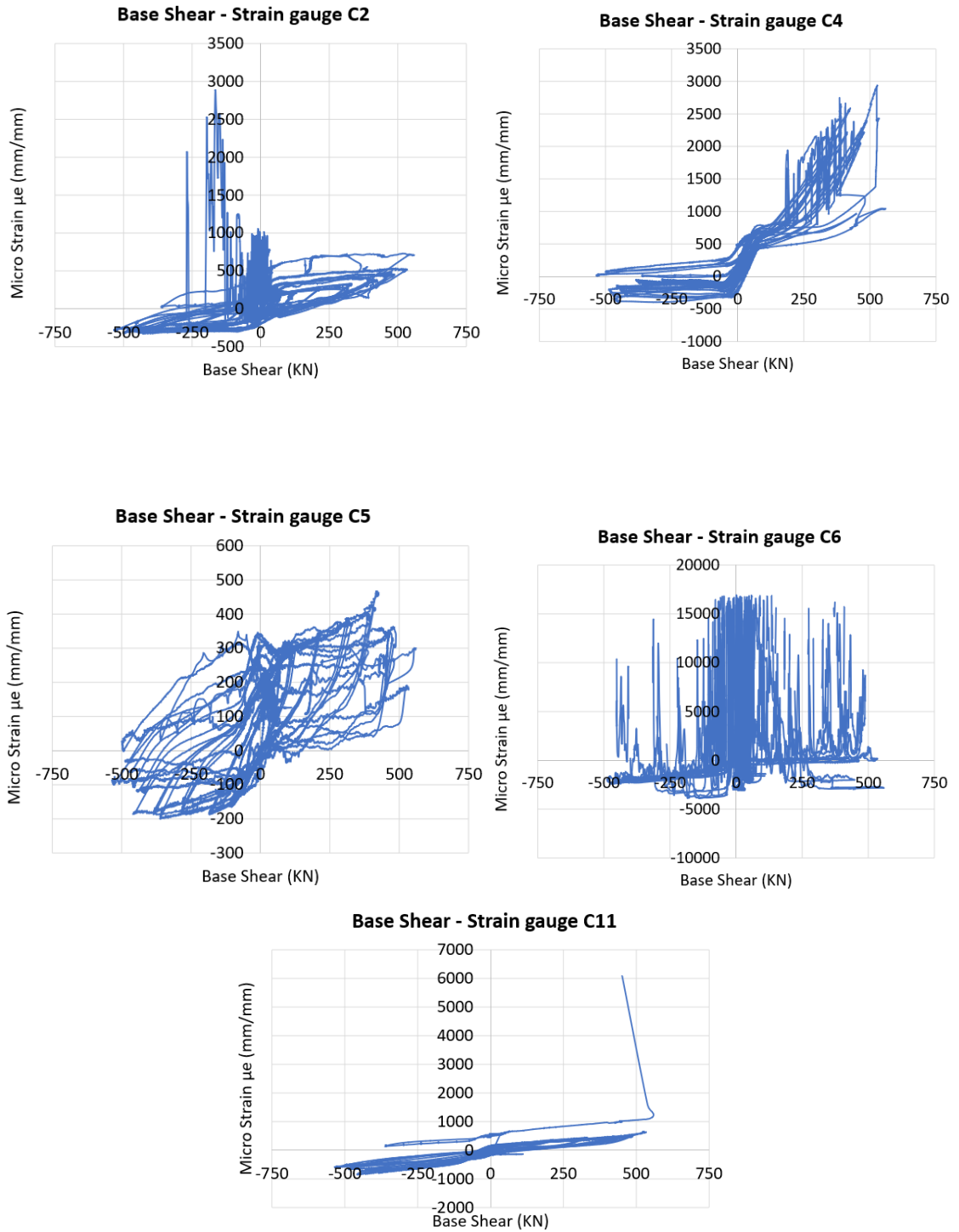


Figure IV-11 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at outer face of base of left the column.

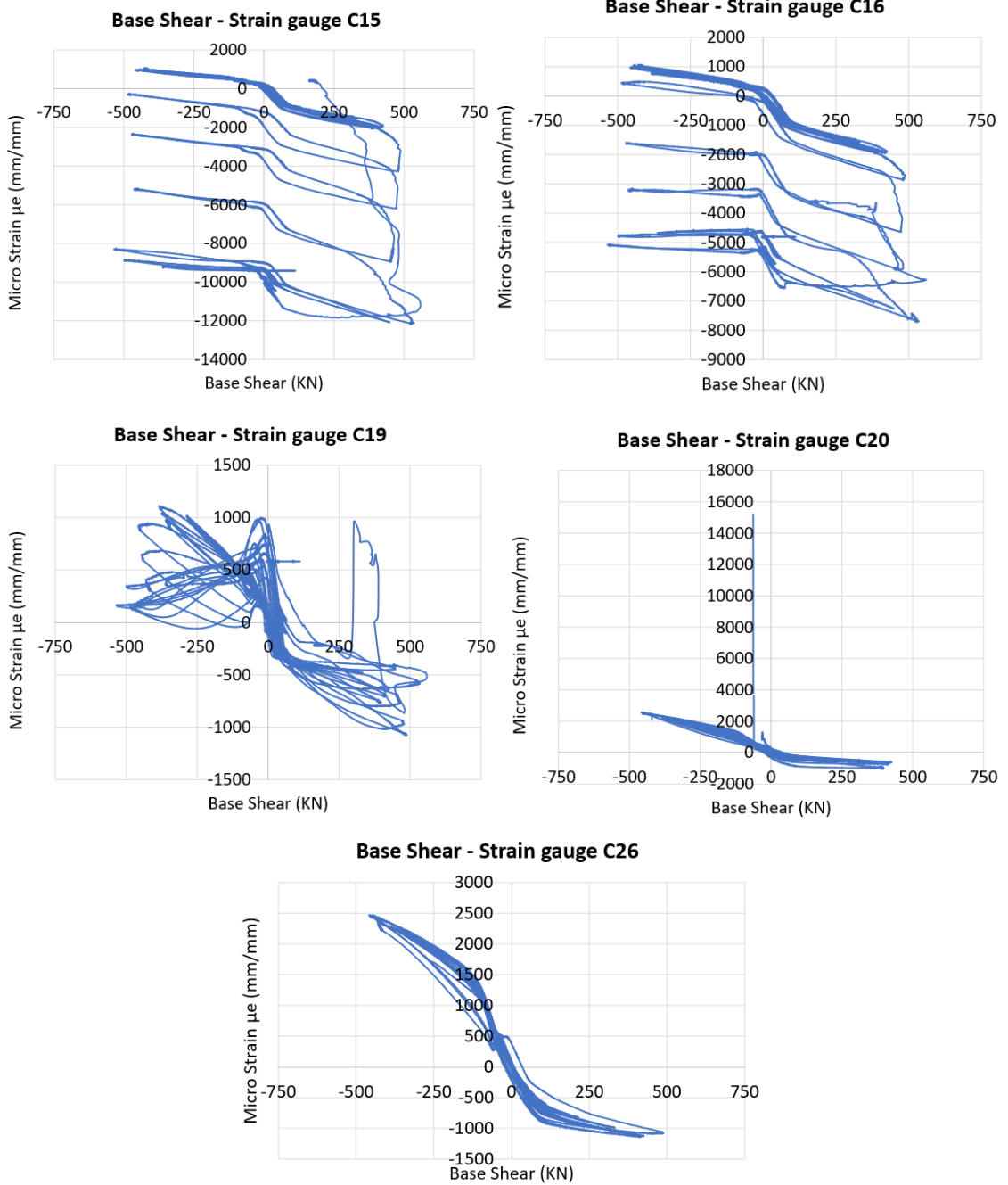


Figure IV-12 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at outer face of base of the right column.

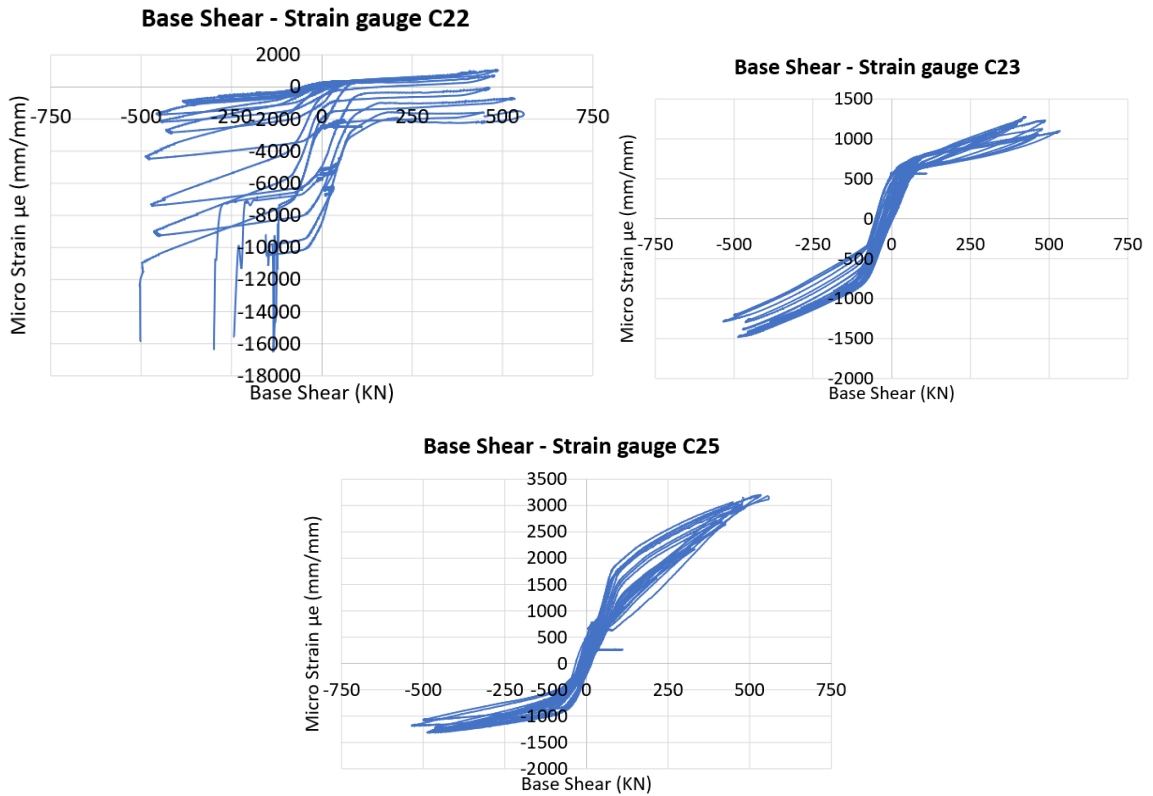
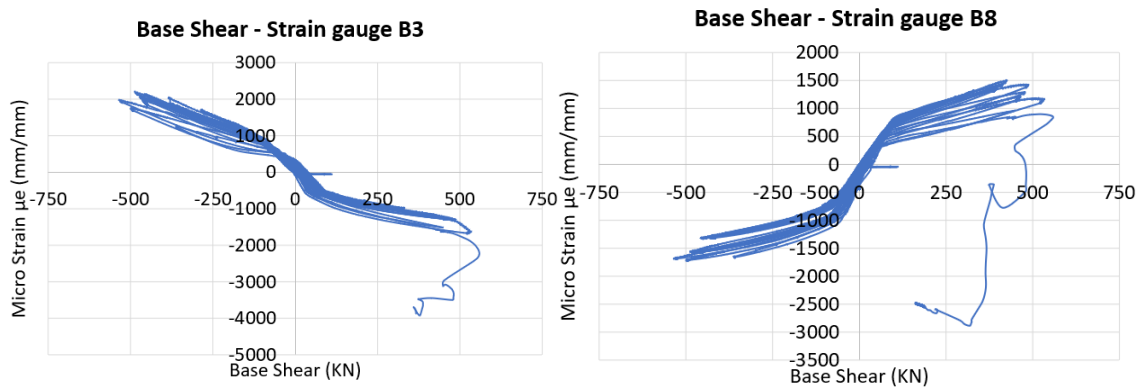


Figure IV-13 Frame lateral displacement vs. strains recorded by strain gauges on the reinforcing bars at inner face of base of the right column.



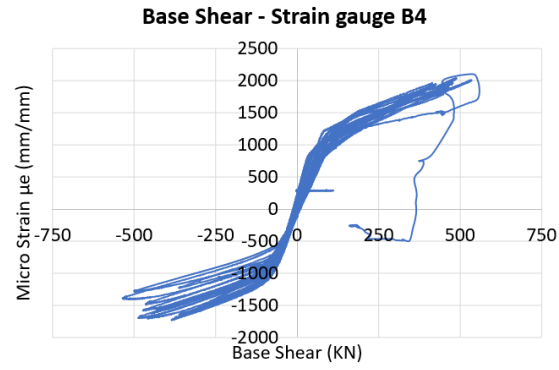


Figure IV-14 Frame lateral displacement vs. strains recorded by strain gauges on the top and bottom reinforcing bars at right side of the beam.

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